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Highway embankments are recommended for consideration as a possible disposal source for discarded solid non-biodegradable waste products. Also, there could be an economic benefit in reduced right-of-way width requirements resulting from steeper embankment slopes, as a result of the stabilizing effects of these waste products.

It is concluded that certain embankment soils could be beneficiated by increased shear strength provided by inclusion of waste materials. Flattened tin and aluminum containers were found to provide the greatest stabilizing benefit and strength improvement. Chopped rubber tire inclusions were beneficial to moderately plastic, silty clay embankment soil with low angle of internal friction. However, this waste product was also found to lower the strength parameters of better quality embankment soils. Broken glass performed as an aggregate, but crushed under heavy loading, thus producing no significant gain or loss in shear strength with the typical embankment soils tested.

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HIGHWAY RESEARCH REPORT

FILL STABILIZATION USING NON-BIODEGRADABLE WASTE PRODUCTS PHASE I

INTERIM REPORT

STATE OF CALIFORNIA BUSINESS AND TRANSPORTATION AGENCY DEPARTMENT OF TRANSPORTATION DIVISION OF HIGHWAYS	TRANSPORTATION LABORATORY RESEARCH REPORT CA-DOT-TL-2124-1-73-25 19-954104

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It is concluded that certain embankment soils could be beneficiated by increased shear strength provided by inclusion of waste materials. Flattened tin and aluminum containers were found to provide the greatest stabilizing benefit and strength improvement. Chopped rubber tire inclusions were beneficial to moderately plastic, silty clay embankment soil with low angle of internal friction. However, this

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waste product was also found to lower the strength parameters of better quality embankment soils. Broken glass performed as an aggregate, but crushed under heavy loading, thus producing no significant gain or loss in shear strength with the typical embankment soils tested.

Additional research encompassing a field study with actual highway embankments is suggested under Phase II to provide more meaningful results using split rubber tires in applications similar to reinforced earth construction.

STATE OF CALIFORNIA—BUSINESS AND TRANSPORTATION AGENCY

RONALD REAGAN, Governor

DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
TRANSPORTATION LABORATORY
5900 FOLSOM BLVD., SACRAMENTO 95819

August 1973

Interim Report (Phase I)
TL No. 652124
FHWA No. D-3-44

Mr. Robert J. Datel
State Highway Engineer

Dear Sir:

Submitted herewith is the interim research report titled:

FILL STABILIZATION USING

NON-BIODEGRADABLE WASTE PRODUCTS

PHASE I

Joseph B. Hannon, P.E.
Co-Investigator

Raymond A. Forsyth, P.E.
Principal Investigator

Very truly yours,

JOHN L. BEATON
Laboratory Director

ACKNOWLEDGMENTS

This study was conducted by the Transportation Laboratory of the California Department of Transportation. Credit should be shared with the personnel involved with data collection and testing, in particular, Mr. David Mauldin who performed the triaxial tests, Mr. Frank Lienert who obtained samples and assisted with the testing, Mr. Ralph Weber who assisted with the drafting, and to Mr. Wesley Gray who supervised the laboratory testing and assisted with the analysis.

The authors would also like to express their appreciation to the following companies for their donation of waste products, in particular, Tires Destructible, Incorporated, of Alviso for the chopped automobile tires, the Los Angeles By-Products Company of Sacramento for the tin and aluminum containers and the Glass Container Corporation of Antioch for the broken glass.

The research work reported herein was accomplished under Highway Planning and Research Project D-3-44 in cooperation with the U.S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the views of the Transportation Laboratory which is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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NOTATION

The following symbols are used in this report:

c	=	Cohesion intercept	in terms of total stress
ϕ	=	Angle of internal friction (Angle of shearing resistance)	in terms of total stress
c, ϕ	=	Shear strength parameters	
σ_d	=	Deviator stress (total stress difference, $\sigma_1 - \sigma_3$)	
σ_1	=	Major principal total stress	
σ_3	=	Minor principal total stress	
UU	=	Unconsolidated - Undrained shear test	
LL	=	Liquid limit	
PL	=	Plastic limit	
PI	=	Plasticity index	

INTRODUCTION

Solid waste poses an increasing threat to the environment. In the United States about 250 billion pounds of solid waste is collected annually. This represents about four pounds per capita per day (10) or probably closer to 5.5 pounds per capita per day with the rate of refuse produced increasing faster than the population (7). However, these estimates may be somewhat low because when all sources including commercial waste are considered, the total per capita waste production is as high as 35 pounds per capita per day (5).

The most frequently used method of solid waste disposal is burial in sanitary landfills. However, this disposal method may result in surface and ground water pollution, if the site is not properly selected or prepared.

Waste utilization or reclamation can be an effective method of disposal. This approach becomes increasingly attractive as our resources become scarcer and our environment more polluted by discarded waste (8). Bargman (2) reported that the City of Los Angeles produces about 430,000 tons of salvagable residential solid waste material annually which could be recycled. "Increasing concern about depletion of our natural resources has focused the country's attention on resource recovery from solid wastes. The dream of 'gold from garbage' is a strong one" (2).

There are many beneficial uses for discarded solid waste. For example, ocean biologists are experimenting with the use of discarded automobile tires as havens for game fish (14). Powdered tire scraps can also be used to enrich the soil and put unproductive land into agricultural use when mixed with chemical salts, fungi or yeasts (15).

The New York State Department of Transportation plans to blend ground-up rubber automobile tires with hot asphalt and use this mixture as a crack and joint sealer (9). The California Department of Transportation is experimenting with a similar application to solve the problem of reflection cracking in asphalt concrete overlays.

Discarded automobile tires represent about 7 percent of the country's solid waste output. It is estimated that about 2 billion discarded tires are scattered around the American landscape and another 200 million are being added each year (9). This waste resource should be utilized, if possible.

Glass is totally inert, and therefore, non-biodegradable. It degrades mechanically similar to natural rock. As an inert construction material, glass can increase the structural strength of various road building elements.

Glass has been experimented with as an aggregate substitute in asphalt concrete. Crushed glass has also been used as an aggregate subbase. On one particular installation, glass consisted of waste material from local glass factories and was processed at a cost equal to that of aggregate (11).

This type of application could also provide a disposal site for various other glass refuse such as no-deposit glass bottles and could offer savings in areas suffering from aggregate shortage.

The objective of the study reported herein was to investigate the feasibility of including selected types of solid non-biodegradable waste material into embankment construction for the purpose of increasing the strength of the soil. This increase would result from a process of mechanical stabilization through reinforcement or a change in the physical characteristics of the soil as in other types of stabilization. This report describes Phase 1 of this study which consisted of a laboratory investigation. Phase 2 will consist of a field study.

Waste materials considered generally non-biodegradable in controlled environments, i.e., not capable of being readily decomposed by biological means especially by bacterial action, were considered in this investigation. These materials consisted of chopped automobile tires, broken glass and flattened tin and aluminum containers. The results of laboratory tests are reported herein. A practical and economically feasible method of embankment stabilization utilizing these selected waste materials could permit steepening of embankment side slopes and could result in significant savings in right-of-way costs. This application could also provide a disposal site for these materials.

CONCLUSIONS

1. When chopped rubber automobile tires are incorporated as a single waste product in an embankment soil a reduction in strength may occur with certain soils. This effect can be offset by mixing this material with other waste products.

2. Chopped rubber tire inclusions could improve the strength properties of moderately plastic clay embankment soils by providing higher strength and the ability to withstand greater deformation (strain) prior to failure.

3. Tin and aluminum waste inclusions can provide significant strength improvement of some embankment soils.

4. In most cases, broken glass will perform as an aggregate or will crush under heavy loading and provide no significant gain or loss in embankment soil strength.

5. The combined mixture of equal amounts of broken glass, chopped rubber tires and tin and aluminum containers provide an averaging effect in terms of balancing the detrimental and beneficial stabilizing aspects of these materials.

6. Waste inclusions provide the maximum stabilizing benefit when mixed with embankment materials with nonplastic fines and high angle of internal friction (ϕ), i.e., in excess of 30 degrees.

7. There appears to be no direct correlation between the effect of waste inclusions on soil strength and the embankment soil's plasticity index, density, cohesion or angle of internal friction.

8. Highway embankments could serve as disposal sites for such waste materials as chopped rubber tires, broken glass and tin and aluminum containers, provided the embankment soils have properties applicable to this type of stabilization.

RECOMMENDATION AND IMPLEMENTATION

It is recommended that this project be extended to Phase 2 and additional research be performed as a field study of various waste stabilized trial embankments. This would provide a more proper and realistic evaluation using a different system of waste placement.

Equipment is available commercially to cut automobile tires in half. These tire halves could be tied end to end and placed in strips or tied in a mat at given intervals within an embankment. This system may provide reinforcement similar to the "reinforced earth" concept and allow steeper embankment construction which would result in reduced right-of-way requirements.

Consideration could also be given to the use of waste inclusions in zoned embankments or possibly in stabilizing berms when the embankment is constructed on a weak foundation. The stabilizing berm, consisting of the compacted soil with the waste inclusions, would serve primarily to provide weight for stability in the outer slopes and also provide a disposal site for non-biodegradable waste materials.

REVIEW OF LITERATURE

Historically, engineers have long been aware of the stabilizing effects of including various materials in earth works. The walls of ancient Roman fortification consisted of alternating layers of earth and brush wood (1). A precise description of this technique is provided in the commentaries of Julius Caesar (3). "All Gallic walls are commonly of this fashion: straight beams are laid together upon the ground at equal intervals of two feet, their inner ends braced together, while along the outer front, the inner spaces are packed with large blocks of stone, and the whole is covered with earth. Upon these is laid a second similar row of beams so that while the same interval is maintained the beams are not contiguous. In this way the whole wall is built up course by course until the full height is maintained." The heavy stones were utilized to resist battering against the outer face. The beams, which were laid perpendicular to the front face, acted as reinforcing elements.

The first disciplined and, by far, the most extensive and successful application by this technique was developed by the French engineer, Henri Vidal (16), in the late 1950's. Vidal's system, known as "reinforced earth", consists of placing steel reinforcing strips at predetermined intervals within the fill mass for the purpose of providing tensile or cohesive strength in a relatively cohesionless material.

Vidal developed the idea for reinforced earth about 15 years ago while visiting a sandy beach on the Mediterranean (4, 6). He toyed with the sand, arranging it in piles, which quickly slid down forming cones with an angle of repose that always remained about the same. He then placed rows of pine needles between layers of sand and discovered that the angle of repose tended more toward the vertical. Essentially he reinforced the sand so that the internal friction between the sand and the pine needles held the sand in place. This theory was verified in 1965 when he designed and built the first reinforced earth embankment in France (6).

In the reinforced earth concept, the steel strip reinforcement resists the forces that develop in the soil mass by means of transfer through friction between the soil grains and the reinforcement. If reinforcement is properly designed and placed, it is possible to avoid shear failure so that the entire mass behaves like a cohesive solid capable of withstanding both internal and external forces.

For a soil to be satisfactory for reinforced earth construction, Vidal suggests that it be composed of granular materials having an angle of internal friction of at least 25 degrees to develop adequate frictional resistance between the soil and the reinforcing material. He also suggests that no more than 15 percent of the soil should be finer than the No. 200 material (16).

One advantage of "reinforced earth" is that it permits the construction of an embankment with vertical side slopes without external restraint as provided by a retaining wall. Thus, the facing serves only to prevent local sloughing and erosion. Within practical limits there is no height limitation except for the bearing capacity of the foundation materials. Also, its flexibility permits construction over foundations which would not normally be suitable for conventional retaining walls. This technique appears to have excellent potential for replacing relatively high retaining structures where soil conditions are suitable.

The first reinforced earth installation in the United States was placed by the California Division of Highways in 1972 on Route 39 in Los Angeles County. Prior to that time the only installations were in France, Canada and Africa (4). Various materials other than steel are also beneficial in providing mechanical stabilization to embankment construction.

A preliminary laboratory investigation reported by Singh and Lee (12) demonstrated that significant improvements in engineering properties of soil can be provided by reinforcing the soil with strands or meshes of materials such as wood, metal, plastic, or glass fibers. These reinforcement materials were 1-inch long thin strips approximately 1/16 inch wide. They were mixed uniformly throughout the soil specimens prior to testing. The addition of 1.3 percent wood shavings was found to increase the strength of dry sand in triaxial testing by as much as 1600 percent. Similar results were also obtained with other reinforcing materials.

Unconfined compressive strength tests on compacted silty clay (LL=38 and PI=21) indicated no significant change in peak strength with the addition of reinforcing material. In some cases the reinforcement reduced the strength but provided the ability to withstand additional deformation prior to failure.

This somewhat confirms the guide criteria suggested by Vidal (16) which recommends the use of soil with an angle of internal friction of at least 25 degrees.

EVALUATION PROCEDURE

This study sought to develop qualitative as well as quantitative answers to the following questions:

1. Would the inclusion of non-biodegradable waste result in a significant improvement in the shear strength properties of an embankment soil?

2. What general type of embankment soil is most responsive to this type of stabilization?

3. What type of waste is most effective (glass, tin and aluminum containers or chopped rubber tires)?

4. What pattern of waste inclusion is most effective?

5. What proportion of waste is necessary to have a significant effect on the shear strength properties of the embankment soil?

6. What change in shear strength properties can be anticipated by this method of stabilization?

7. What effect will this change in shear strength have on the side slope requirements of an embankment?

To provide answers to these questions, samples of four broadly represented embankment soils in California were selected for laboratory testing. These soils ranged from SM (silty sand) to CL (gravelly and sandy clay) and consisted of samples of sufficient quantity recently on hand from other studies or readily obtainable from field sources.

The material from Source 1 (WF-1) is a brown, slightly plastic, clayey silt and is classified as ML according to the Uniform Soil Classification System. This source is located at Discovery Park at the confluence of the Sacramento and American Rivers in Sacramento.

Source 2 (WF-2) is a brownish red, nonplastic silty sand classified as an SM soil and also identified as Aromas Red sand. This material was sampled from a source near Prunedale, north of Salinas, California.

Source 3 (WF-3) is a tan, moderately plastic, clayey silt classified as a CL soil and is identified in this test program as Altamont Pass #8. This source is located near Altamont, California.

Source 4 (WF-4) is a tan, moderately plastic, slightly silty clay classified as a CL soil. In this study it is identified as Altamont Pass #6. This source is also located in the Altamont area west of Tracy, California.

All soil samples were subjected to various index tests: grading, Atterberg Limits, R-Value and impact compaction.

Grading curves for these materials are presented on Figure 1. Classification and other index test results are shown in Table 1.

Shear strength testing was conducted according to Test Method No. Calif. 232 titled, "Method of Test for Large Scale Triaxial Compression of Soils" using the high pressure triaxial chamber of the Transportation Laboratory of the California Department of Transportation. Test specimens were 12 inches in diameter by 27 inches high. A strength comparison of the control specimens is presented on Figure 2. For sample preparation and detailed test procedure refer to the Appendix of this report.

Single specimen "bump" testing as described by Smith and Kleiman (13) in 1970 was utilized to develop Mohr strength envelopes for these materials. This single specimen multiple procedure minimizes labor costs and eliminates individual specimen variation. Reference 13 presents justification for single specimen multiple triaxial tests. A comparison of the test results produced by this procedure with that produced by conventional multiple specimen triaxial tests is presented on Figures 3a and 3b for the Discovery Park (WF-1) soil.

Unconsolidated-Undrained (UU) shear tests were performed on all specimens compacted to 90% of a laboratory standard impact density at optimum moisture content prepared according to Test Method No. Calif. 216. Compaction curves for the four soil sources are presented on Figures 4a to 4d. Maximum density and optimum moisture content results are presented on Table 1.

High embankment conditions were selected to evaluate the stabilizing effects of non-biodegradable waste inclusions in order that the resulting benefication or weakening would not be masked out by experimental error. To represent embankment soils under these conditions in the laboratory, initial confining pressures of 100, 200, and 400 psi (7.2, 14.4 and 28.8 TSF) were selected.

The procedure consisted of applying and maintaining the initial confining pressure (100 psi) and then increasing the vertical deviator stress until the specimen reached 6 percent axial strain. The vertical deviator stress was then reduced to zero and the specimen was allowed to rebound. The next increment of confining pressure (200 psi total) was then applied, i.e., the specimen was "bumped" to the next load increment up to 12 percent axial strain. This process was repeated at 12 percent strain (400 psi confining pressure) and then loaded vertically to failure.

The initial series of triaxial shear strength tests were scheduled for the soil which, according to French experience, would provide the greatest response to stabilization through reinforcement, i.e., the soil with the higher angle of internal friction. However, some variation from this scheduling was necessary due to the availability of materials at the start of the testing period. Therefore, the locally available Discovery Park soil (WF-1) was initially utilized for the test program. This soil is somewhat of a midrange material in terms of internal angle of friction (24 degrees) and therefore borderline in response to stabilization according to the French criteria. Triaxial tests were conducted to determine the:

1. Shear strength of the untreated (unreinforced) soil.

2. Shear strength of soil specimens treated with layered placement of tin and aluminum containers, broken glass, chopped rubber automobile tire particle inclusions and a mix of these items.

A layered system of waste placement was selected because better field control could be achieved for this type of embankment construction than for random placement of waste and soil.

Soil specimens 12-inches in diameter and 27-inches high were fabricated with waste inclusions placed in 1 to 4 layers within the specimens. This simulated layers of soil placed in the field in 7-8 inch lifts and waste material end dumped in 2-inch $\pm$ layers. Thicker waste layers were avoided due to the difficulty in filling voids between the waste particles with soil. Consideration was also given to resilience and compressibility in establishing a unit layer thickness for the waste inclusions.

Figure 5 presents a typical soil specimen with the location of 2 layers of waste material.

Optimum percentages of waste and the optimum layering systems were selected for the other three embankment soils, based upon the results of this initial test series. However, additional tests with different waste inclusion patterns were also performed to provide more information.

Chopped rubber from automobile tires was donated by Tires Destructible, Incorporated located in Alviso, California. This sample was graded in the laboratory to determine the distribution of the various sizes. Since the maximum aggregate size is 3-inches for the 12-inch diameter specimen, rubber particle inclusions were limited to those passing through a 3-inch screen. The sample grading of this material is presented in Table 2.

To minimize soil specimen resilience, a maximum layer thickness of about 2-inches was used for the rubber inclusions. The total amount of rubber per specimen was based on a percentage of the soil specimen dry weight and varied from 2 to 10.6 percent.

The second waste material used in the investigation consisted of broken glass containers donated by Glass Container Corporation located in Antioch, California. This was used in about the same percentages as the rubber tire particles with a maximum size of 3-inches.

The third waste material which was investigated, consisted of tin and aluminum beverage and food containers which were donated by the Los Angeles By-Products Company of Sacramento.

In preparing this material for use, it was first flattened and passed through a 3-inch screen opening. This material was then placed in a layering system consisting of two to four layers of flattened tin and aluminum per unit layer within the soil specimen. This waste was used in percentages similar to that of the other waste materials.

The waste mix consisted of equal percentages of the above materials. These were used in an optimum amount and optimum layering as determined from the initial test series.

DATA COLLECTION AND ANALYSES

The evaluation of the stabilizing benefit of incorporating waste inclusions in soil specimens is presented below by soil type with resulting data and analyses included.

Figures 6, 9, 12 and 15 present the effect of waste type and percentage of waste inclusions on deviator stress. Here, a gain or loss in deviator stress at 6% strain is compared to the compacted soil without waste inclusions, which was used as the untreated control specimen. The effect of the layering pattern of the waste inclusions is also shown.

Figures 7, 10, 13 and 16 show the stress-strain relationship for the treated and untreated soil specimens.

Figures 8, 11, 14 and 17 present the effect of waste type on the shear strength parameters (ϕ and c). The strength parameters are compared graphically on the Mohr diagram and are also shown in tabular form.

Test Series WF-1

With the exception of the test specimen with 2 layers of flattened cans, (Figure 6) all other specimens failed to attain a deviator stress equivalent to the untreated control embankment soil at 6 percent strain.

The chopped rubber tire inclusions produced a significant loss in deviator stress (24% with 10.6% rubber by weight). There was less than 4% loss in deviator stress produced by the inclusion of 6.4% flattened cans. However, when a total of 8.7% cans were included in 4 layers, the loss in deviator stress was reduced to less than 1%. When broken glass was added at 12% by weight of dry soil, crushing developed and a minimal effect on deviator stress was noted at 6 percent strain.

The detrimental effect of the rubber appeared to be offset when combined in a mix of equal portions of flattened cans and broken glass. The number of waste layers per specimen had some effect on this test series but was not considered significant.

Figure 7 indicates an increase in deviator stress for all treated specimens at strain levels in excess of 18 percent with a very significant increase noted for specimen (WF-1-11) with flattened can inclusions. Photo 1 shows this specimen after triaxial testing with the 3 layers of flattened can exposed.

Test Series WF-2

Figure 9 indicates a significant loss in deviator stress resulting with chopped rubber tire inclusions at 6 percent strain. The magnitude of this strength loss was similar to that of the WF-1 soil. However, both the cans and the broken glass inclusions produced a gain in deviator stress. A maximum gain of over 28% was produced using 6.3% cans. Again, the loss in deviator stress due to the rubber was offset by the other waste products for the specimens with the mixed waste inclusions. There was no significant difference in the effect between a 2- and a 3-layer waste placement.

A consistent decrease in deviator stress with increasing strain levels is suggested by Figure 10 for specimen (WF-2-4) with 6.8% chopped rubber inclusions. Photo 2 shows a cross-section of this specimen after triaxial testing. Figure 10 also suggests a significant gain in deviator stress for the specimen treated with 6.3% flattened cans. However, a slight loss in deviator stress was produced with 9.3% glass inclusions (specimen WF-2-10) at strain levels in excess of 18 percent. Photo 3 shows the typical grading of the broken glass prior to testing and also the crushed condition following triaxial testing of specimen WF-2-10. This specimen performed similar to the untreated soil.

Figure 11 indicates that this embankment soil produced a ϕ angle of 33 degrees for the untreated control condition. A reduction in the ϕ parameter resulted from the addition of chopped rubber and was of similar magnitude to that produced with the WF-1 embankment material. The flattened cans produced up to 8 degrees increase in ϕ angle. However, the maximum strength was not defined at 6.3% cans as was the case with the deviator stress (Figure 9). Photo 4 shows a shear plane that developed during failure of specimen WF-2-7 with 4.1% flattened can inclusions. Photo 5 shows the sheared metal cans which were removed from the failure plane of this specimen.

Test Series WF-3

As indicated by Figure 12 all waste inclusions produced a loss in deviator stress at 6% strain. However, broken glass was not tested as a single item during this series due to its insignificant effect and also due to the possible reoccurrence of membrane rupture as in series WF-2.

The chopped rubber produced a significant loss in deviator stress (35% with 6.7% rubber) with no significant difference between 2 and 3 layers of waste as indicated by the projected line through the 2-layer specimen on Figure 12.

Figure 13 suggests that for strain levels in excess of 10 percent, a gain in deviator stress occurs with 6.4% flattened can inclusions (WF-3-2). This gain in deviator stress increased the ϕ angle by 3 degrees for this material (Figure 14). A 4.5 degree loss in ϕ angle was produced by 6.7% chopped rubber. The mixed waste produced a loss of only 1 degree.

Test Series WF-4

Figure 15 indicates that rubber inclusions are not as detrimental to this embankment soil as to those previously discussed for the 6% strain level. Figure 16 suggests that the maximum deviator stress actually increased by 13 percent with the addition of 6.7% rubber.

The effect of broken glass was found to be similar to the rubber at 6% strain but achieved a higher deviator stress at strain levels in excess of 12%. The gain in deviator stress with flattened can waste was not as significant with this embankment soil. Mixed waste produced a slight gain in deviator stress at 6% strain and the effect was similar to that of rubber at the higher strain levels.

The control soil produced a ϕ angle of 14 degrees and a cohesion intercept of 27 psi as indicated by Figure 17 (Envelope 1). Photo 6 shows the failure plane that developed in the untreated soil specimen. In all cases the strength parameter ϕ was increased by the addition of waste products, however, the c parameter was slightly reduced. This slight change in the c parameter is offset by the increased ϕ angle for normal stresses over 100 psi as represented by this investigation. Therefore, a stabilizing benefit would be provided by adding waste inclusions to this soil for embankments with height in excess of 120 feet.

Summary

No definite correlation could be defined between the relative effect of waste inclusions and the properties of the embankment soil. However, the maximum stabilizing benefit was developed with the nonplastic soil having the highest angle of internal friction (WF-2 with $\phi = 33°$).

Flattened tin and aluminum containers provided the greatest stabilizing benefit and soil strength improvement of all waste products tested.

Chopped rubber inclusions were beneficial to the moderately plastic, silty clay embankment soil (WF-4) with a low angle of internal friction (14°) and a cohesion intercept of 27 psi.

This waste material provided the embankment soil with higher strength and the ability to withstand greater deformation prior to failure. This property was also noted for the slightly plastic clayey silt embankment soil (WF-1) at the high strain levels.

Broken glass performed as an aggregate, but crushed under heavy loading, thus developed no significant gain or loss in shear strength properties for the embankment soils tested.

Triaxial testing appeared to represent the effects of chopped rubber inclusions on soil strength. However, the evaluation of whole or half tire units will require full-scale field testing.

In testing the flattened tin and aluminum cans, size effects may not have been fully accounted for in the laboratory. This may have some influence on field results.

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13. Smith, Travis and Kleiman, William, "Behavior of High Earth Embankments on U.S. 101, HRB Record No. 345, 1971, pp. 90-110.

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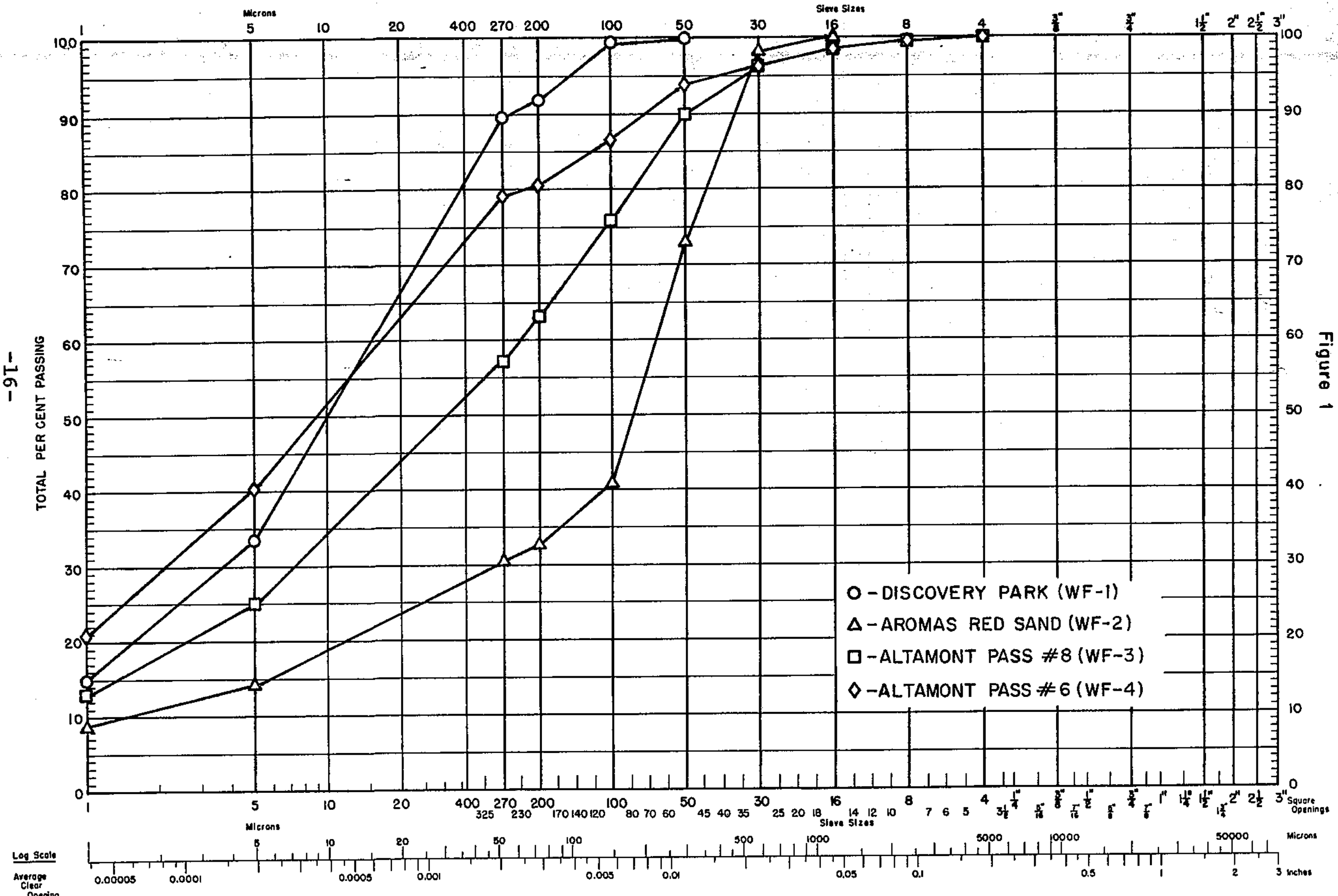
STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS — DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

GRADING ANALYSIS



A.A.S.O. Designation — M92-42. A.S.T.M. Designation — E11-39. Adopted by State — 1940

Figure 1

Figure 2

STRENGTH COMPARISON
OF CONTROL SPECIMENS

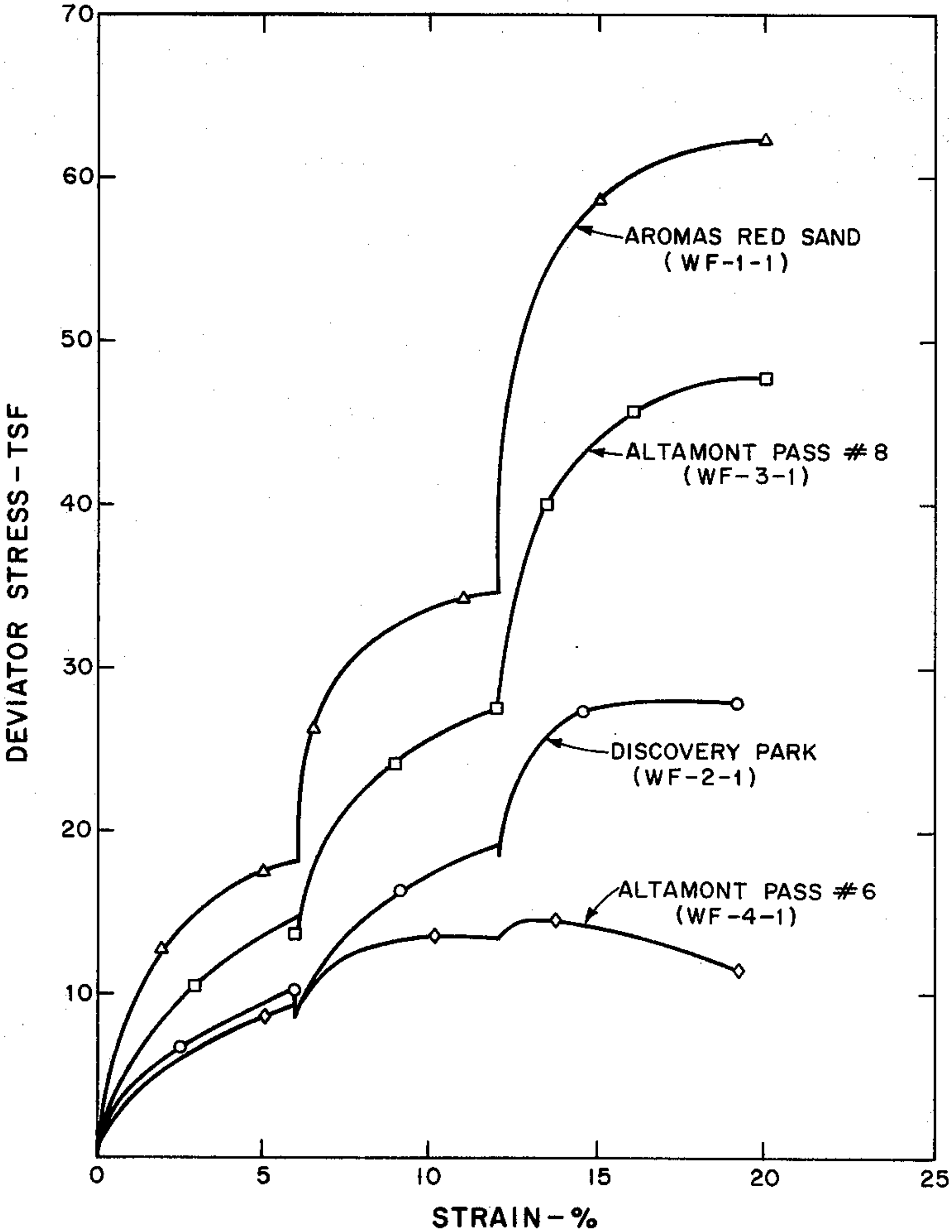


Figure 3

COMPARISON OF CONVENTIONAL AND MULTIPLE STAGE TRIAXIAL TESTS (WF-1)

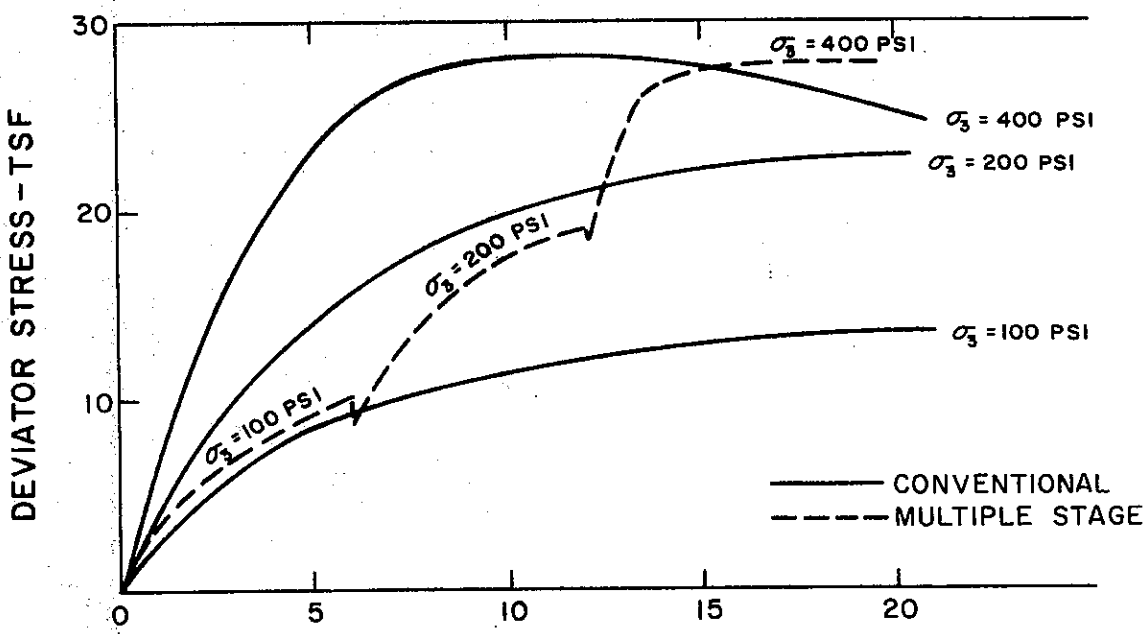


Figure 3 a.

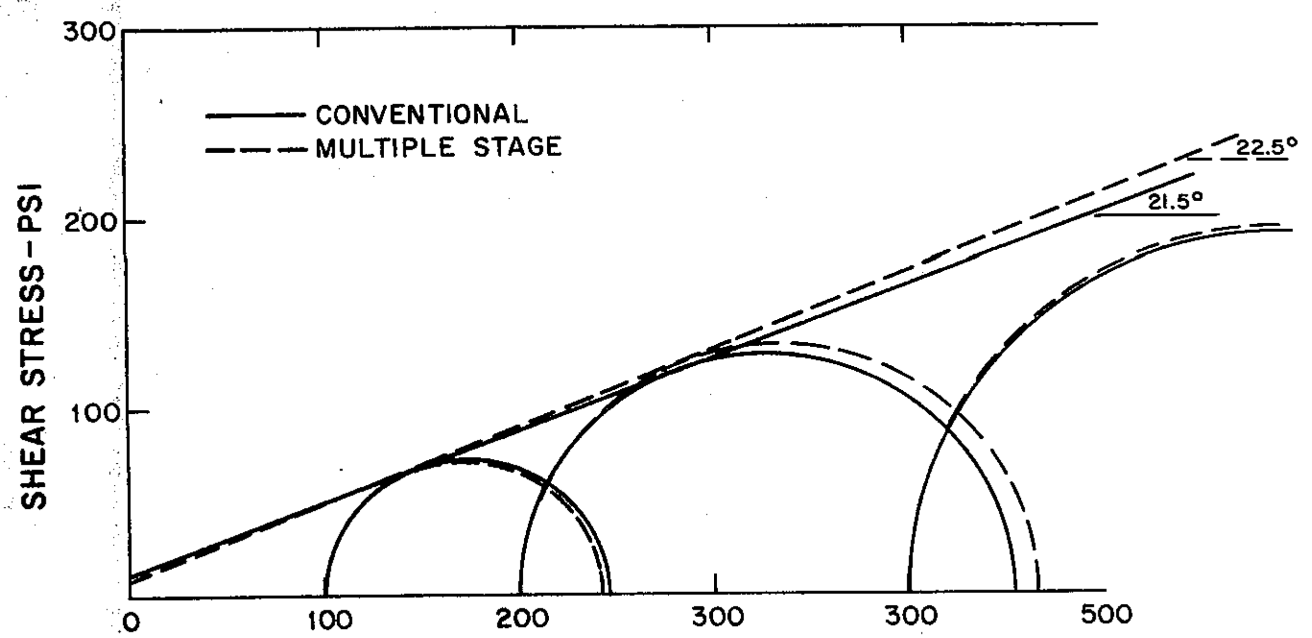


Figure 3 b.

COMPACTION CURVES DETERMINED BY CALIFORNIA IMPACT PROCEDURE

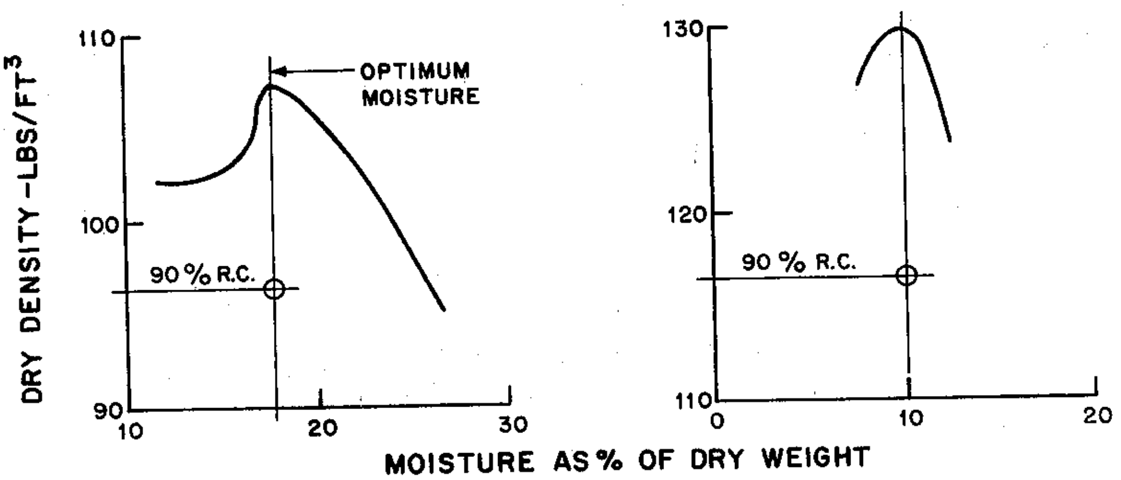


Figure 4a. DISCOVERY PARK WF-1 Figure 4b. AROMAS RED SAND WF-2

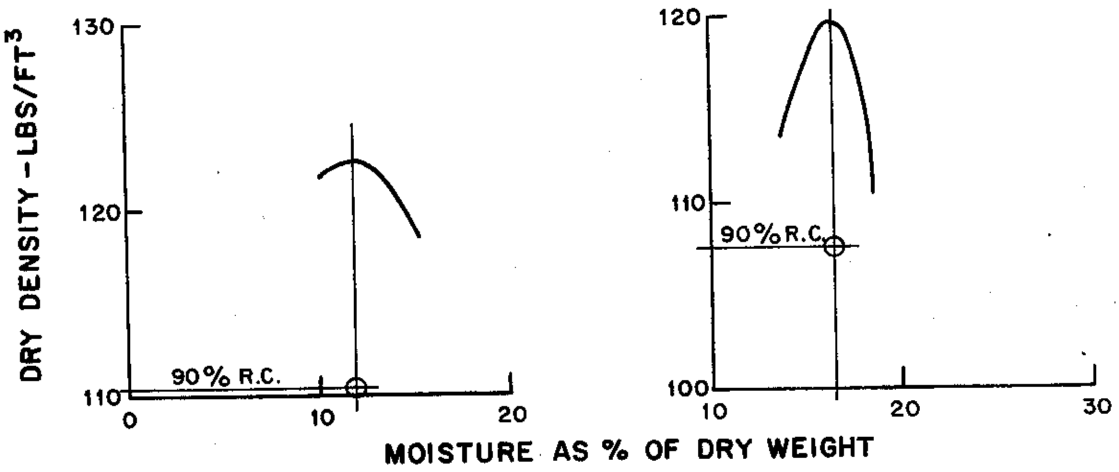
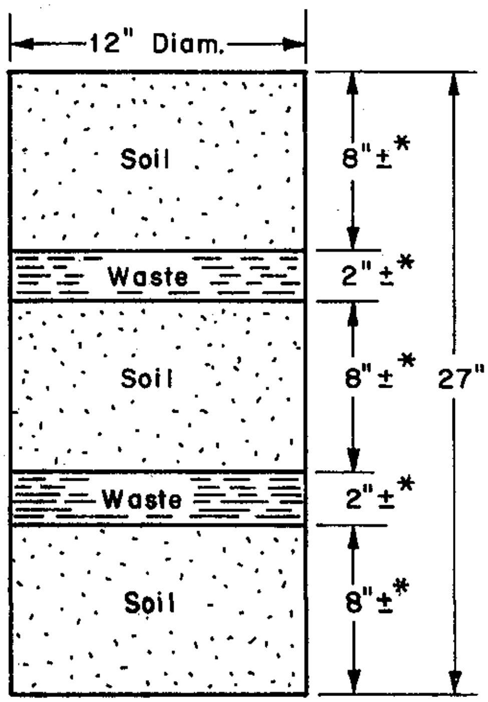


Figure 4c. ALTAMONT #8 WF-3 Figure 4d. ALTAMONT #6 WF-4

⊕ = DESIGN VALUE

Figure 5

TYPICAL TREATED SOIL SPECIMEN



* Approximate Thickness

Figure 6

EFFECT OF WASTE INCLUSIONS ON DEVIATOR STRESS AT 6% STRAIN

DISCOVERY PARK (WF-1)

TREATMENTS RELATED TO CONTROL TEST SPECIMEN
WITH DEVIATOR STRESS (σd) AT 6%
STRAIN = 10.5 TSF

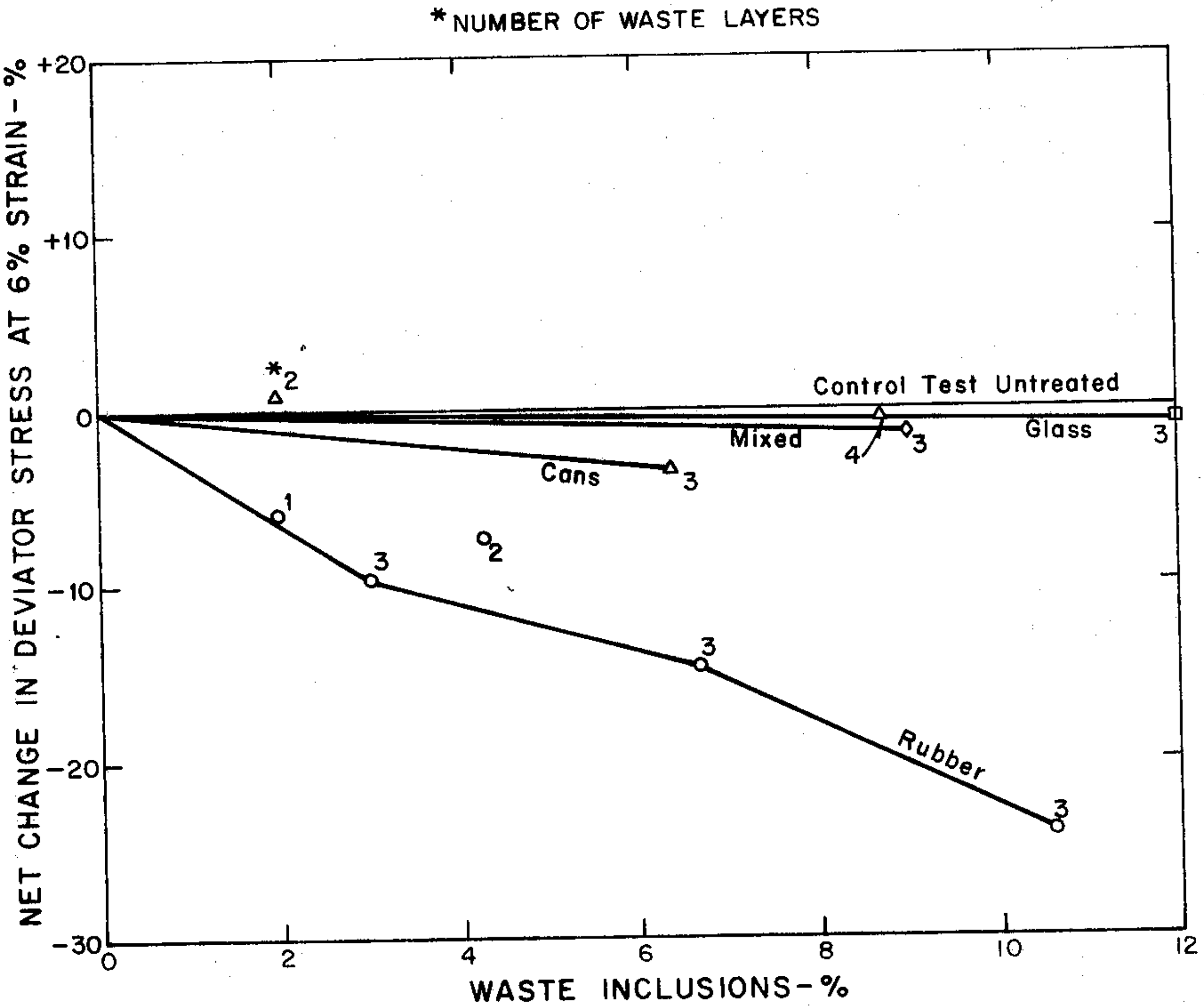


Figure 7

STRESS-STRAIN RELATIONSHIP
FOR TREATED AND UNTREATED SOIL
DISCOVERY PARK SOIL

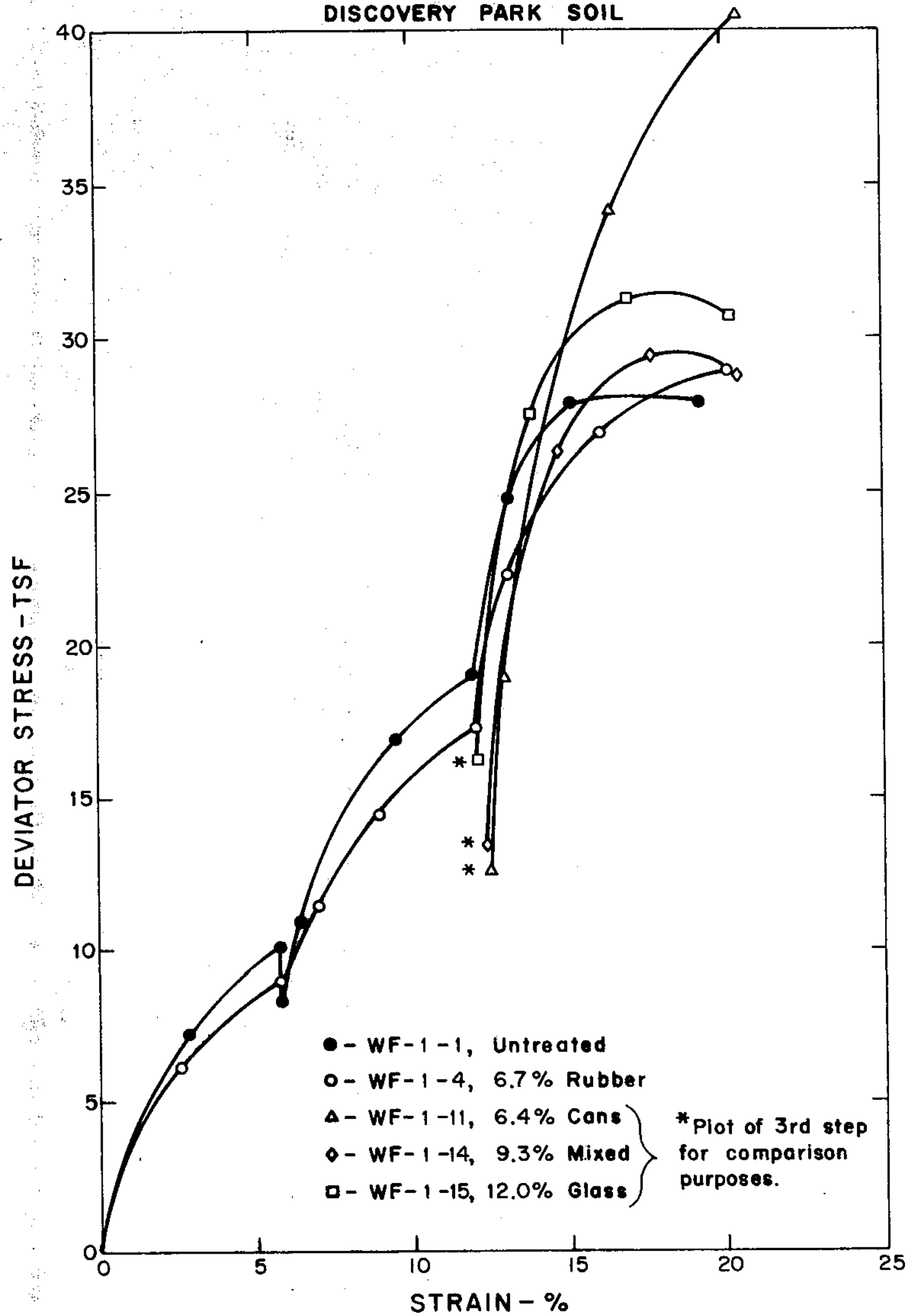
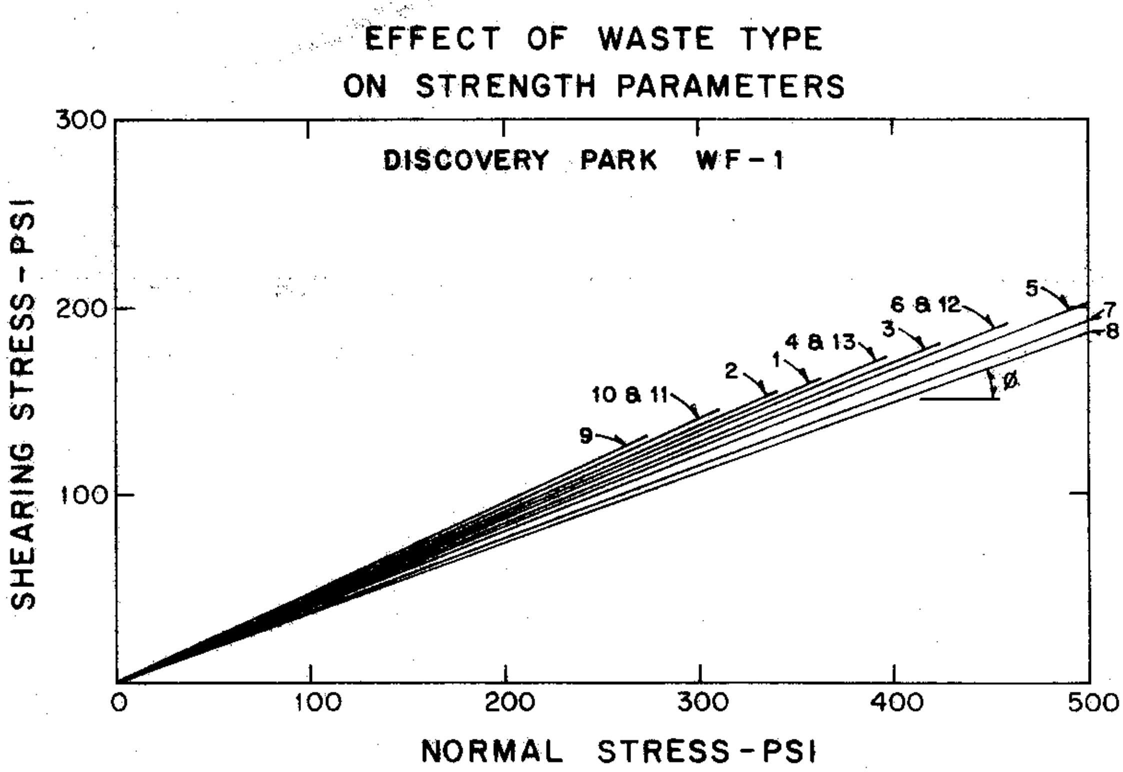


Figure 8



Envelope	Type of Waste	Percent Waste (% Dry Wt. of Soil)	Number of Layers	After Test Moisture (%)	c (PSI)	Ø (Deg)	Test Number
1	-	-	-	17.4	0	24.0	WF-1-1
2	-	-	-	16.6	0	24.5	WF-1-5
3	-	-	-	17.4	3	23.0	WF-1-6
4	Rubber	2.0	1	17.5	0	23.5	WF-1-9
5	"	3.1	3	16.9	3	22.0	WF-1-3
6	"	4.3	2	17.6	3	22.5	WF-1-8
7	"	6.7	3	17.4	3	21.0	WF-1-4
8	"	10.6	3	17.4	0	20.5	WF-1-7
9	Cans	4.2	2	17.0	0	25.5	WF-1-13
10	"	6.4	3	17.2	0	25.0	WF-1-11
11	"	8.7	4	16.9	0	25.0	WF-1-12
12	Mixed	9.0	3	17.6	0	22.5	WF-1-14
13	Glass	12.0	3	17.4	0	23.5	WF-1-15

Figure 9

EFFECT OF WASTE INCLUSIONS ON DEVIATOR STRESS AT 6% STRAIN

AROMAS RED SAND (WF-2)

TREATMENTS RELATED TO CONTROL TEST SPECIMEN
WITH DEVIATOR STRESS (σd) AT 6%
STRAIN = 17.5 TSF

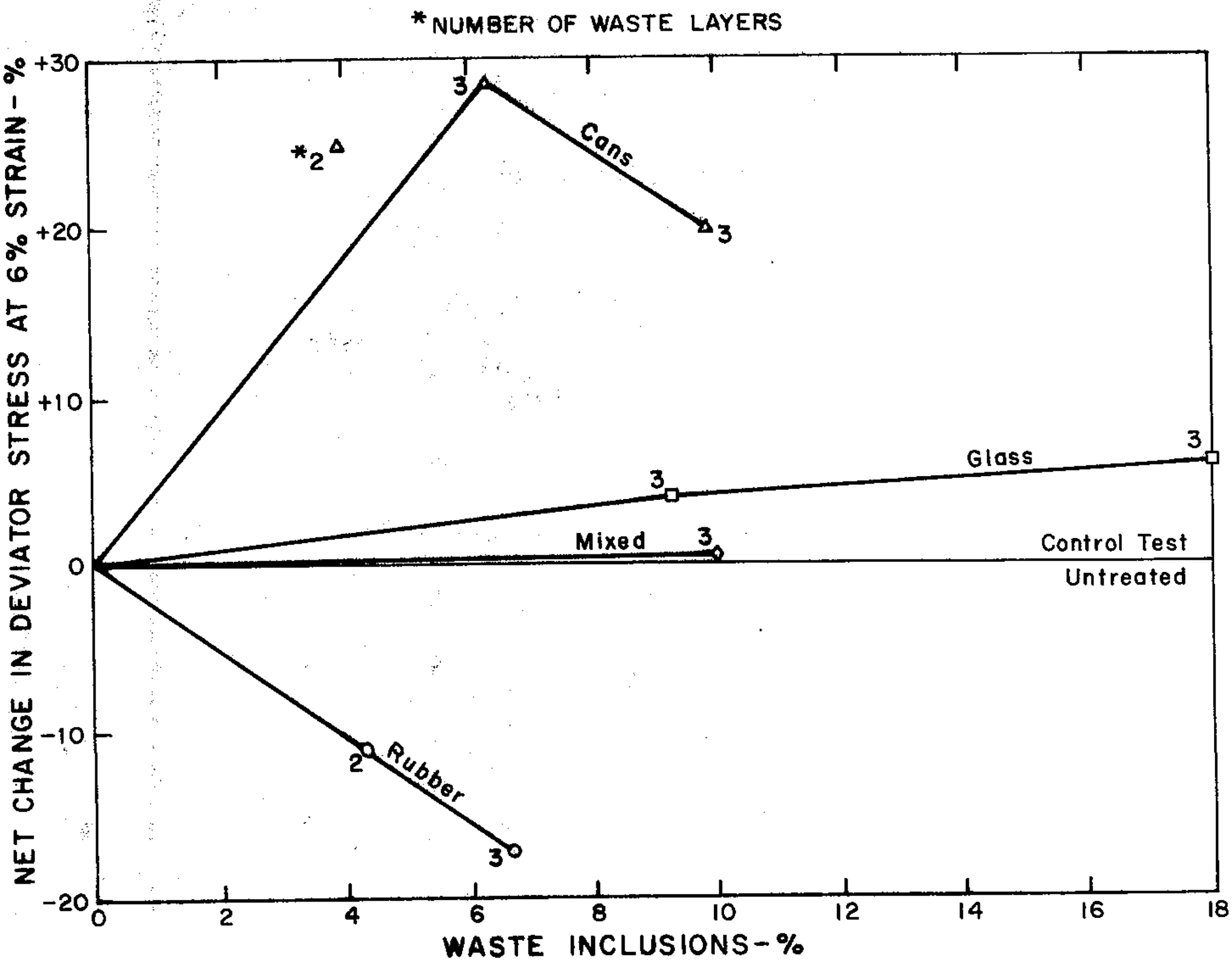


Figure 10

STRESS - STRAIN RELATIONSHIP
FOR TREATED AND UNTREATED SOIL
AROMAS RED SAND

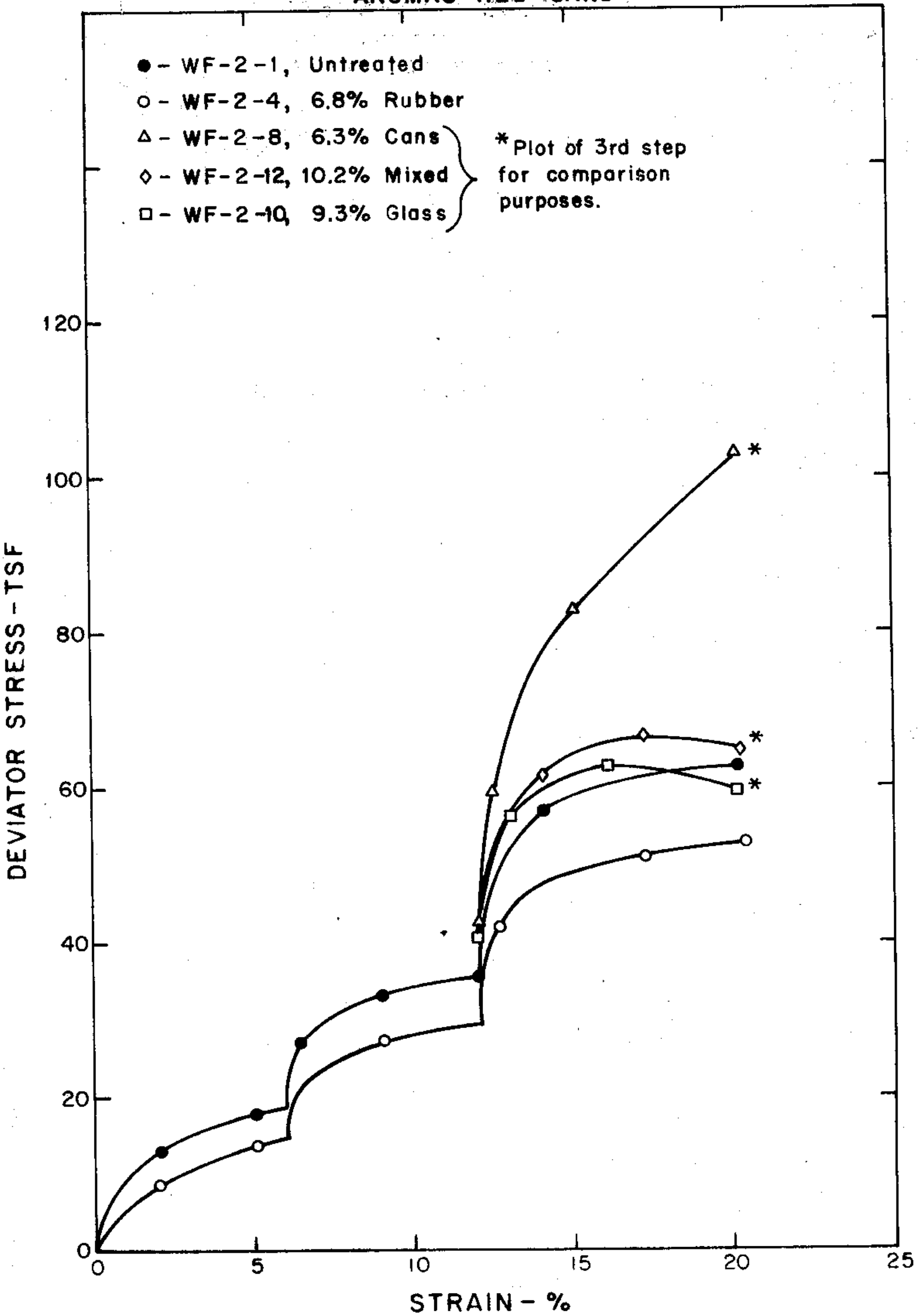
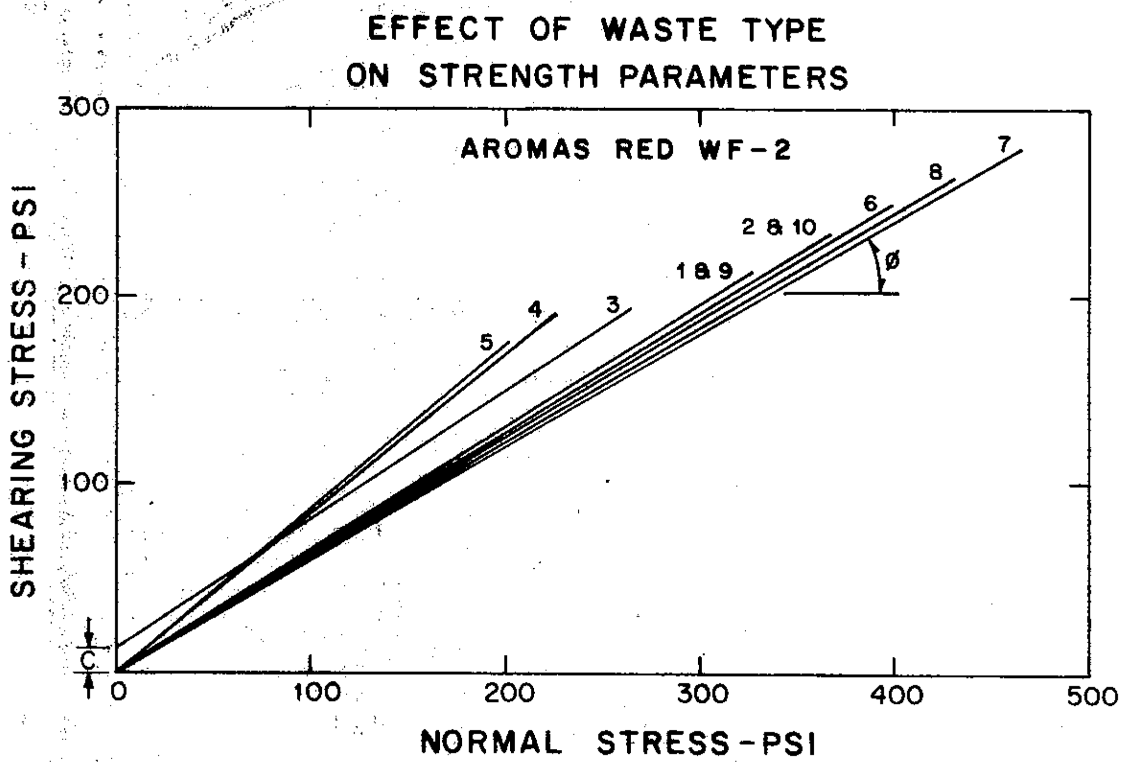


Figure 11



Envelope	Type of Waste	Percent Waste (% Dry Wt. of Soil)	Number of Layers	After Test Moisture (%)	c (PSI)	Ø (Deg)	Test Number
1	-	-	-	9.8	0	33.0	WF-2-1
2	-	-	-	9.9	0	32.5	WF-2-3
3	Cans	4.1	2	9.6	14	34.0	WF-2-7
4	"	6.3	3	9.8	0	40.0	WF-2-8
5	"	9.8	3	10.6	0	41.0	WF-2-9
6	Rubber	4.3	2	9.4	0	32.0	WF-2-5
7	"	6.8	3	10.1	0	30.0	WF-2-4
8	Glass	9.3	3	9.8	0	31.5	WF-2-10
9	"	18.0	3	10.7	0	33.0*	WF-2-11
10	Mixed	10.2	3	9.8	0	32.5	WF-2-12

*Estimated - Membrane ruptured at 7.5%

Figure 12

EFFECT OF WASTE INCLUSIONS ON DEVIATOR STRESS AT 6% STRAIN

ALTAMONT PASS #8 (WF-3)

TREATMENTS RELATED TO CONTROL TEST SPECIMEN
WITH DEVIATOR STRESS (σ_d) AT 6%
STRAIN = 14.79TSF

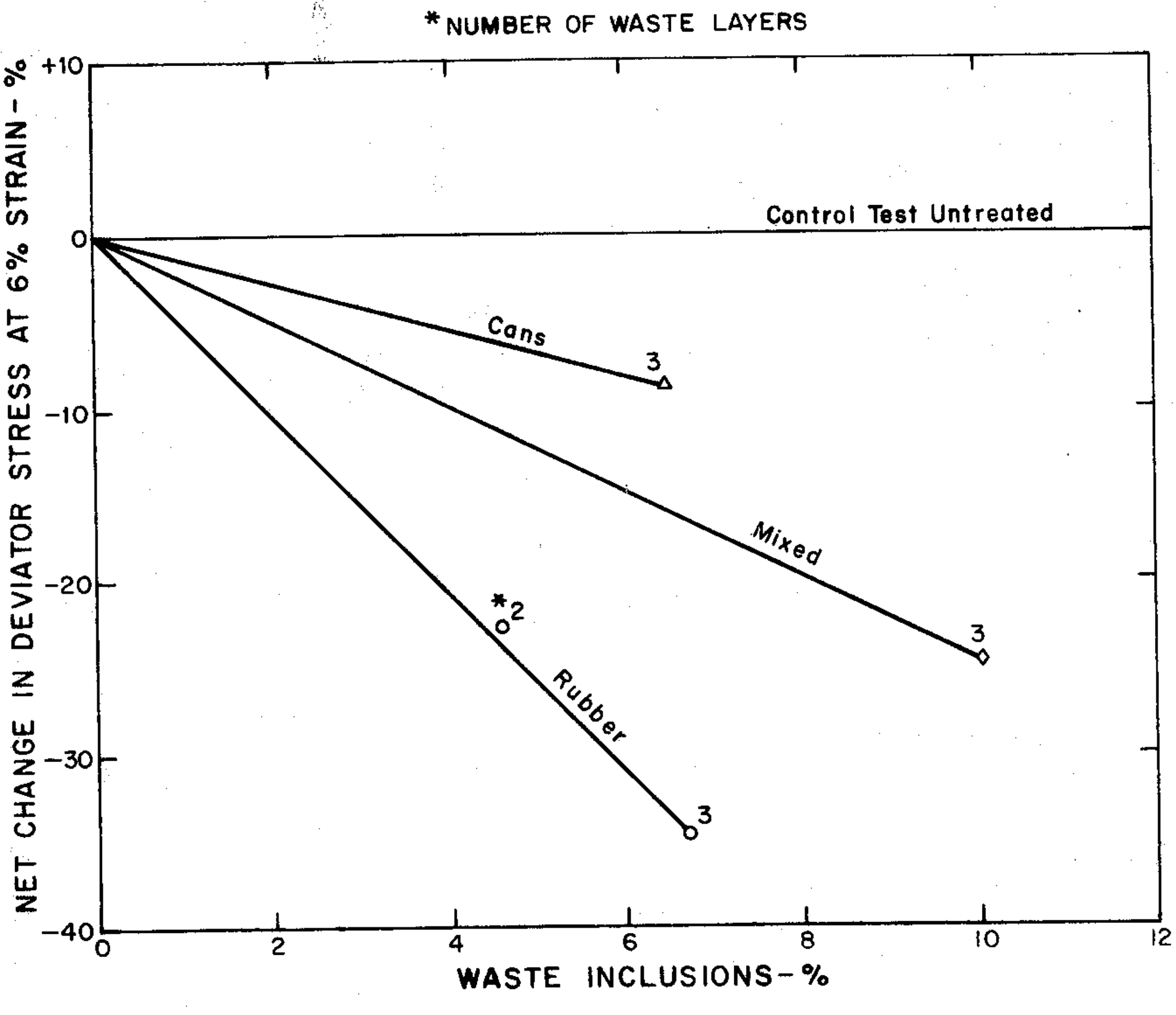


Figure 13

STRESS – STRAIN RELATIONSHIP
FOR TREATED AND UNTREATED SOIL

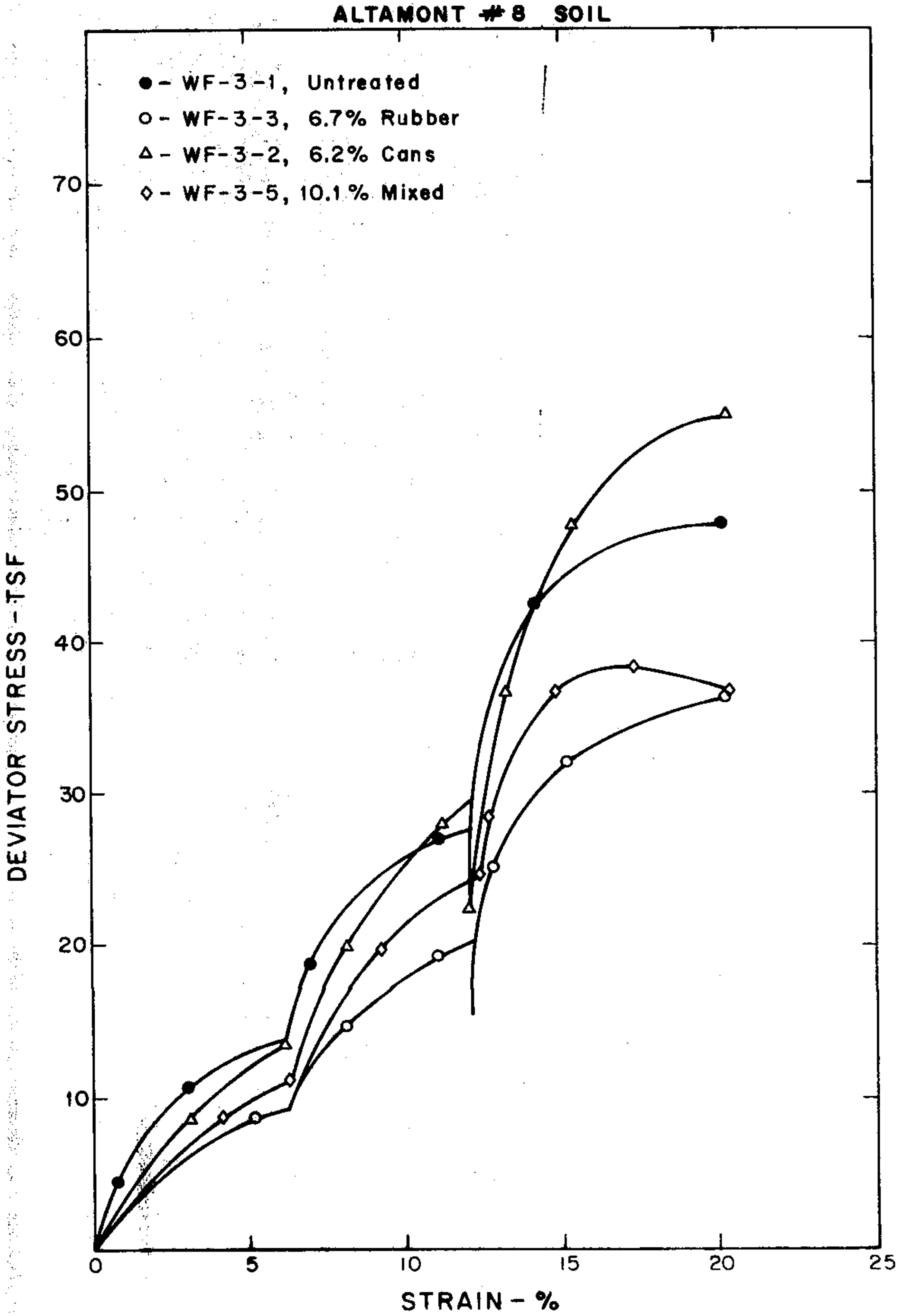
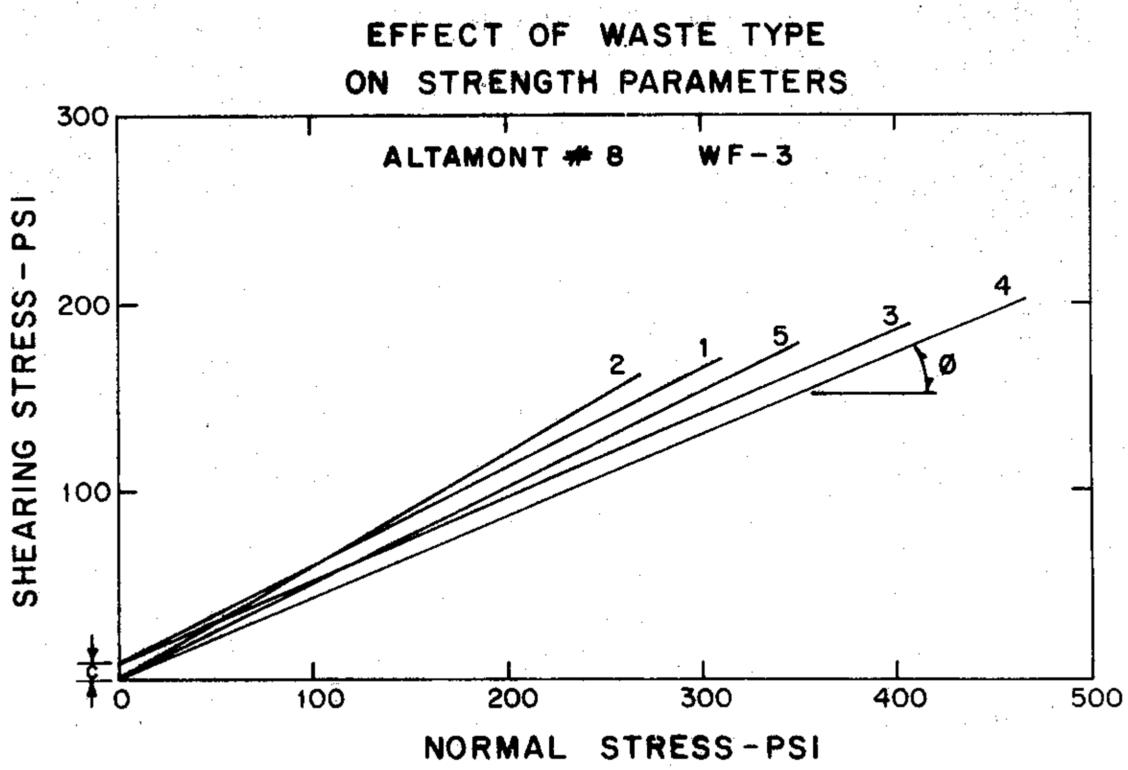


Figure 14



Envelope	Type of Waste	Percent Waste (% Dry Wt. of Soil)	Number of Layers	After Test Moisture (%)	c (PSI)	Ø (Deg)	Test Number
1	-	-	-	11.2	5	28.0	WF-3-1
2	Cans	6.4	3	11.8	0	31.0	WF-3-2
3	Rubber	4.6	2	11.0	7	24.0	WF-3-4
4	"	6.7	3	11.3	0	23.5	WF-3-3
5	Mixed	10.1	3	11.8	0	27.0	WF-3-5

Figure 15

EFFECT OF WASTE INCLUSIONS ON DEVIATOR STRESS AT 6% STRAIN

ALTAMONT PASS #6 (WF-4)

TREATMENTS RELATED TO CONTROL TEST SPECIMEN WITH DEVIATOR STRESS (σ_d) AT 6% STRAIN = 9.33 TSF

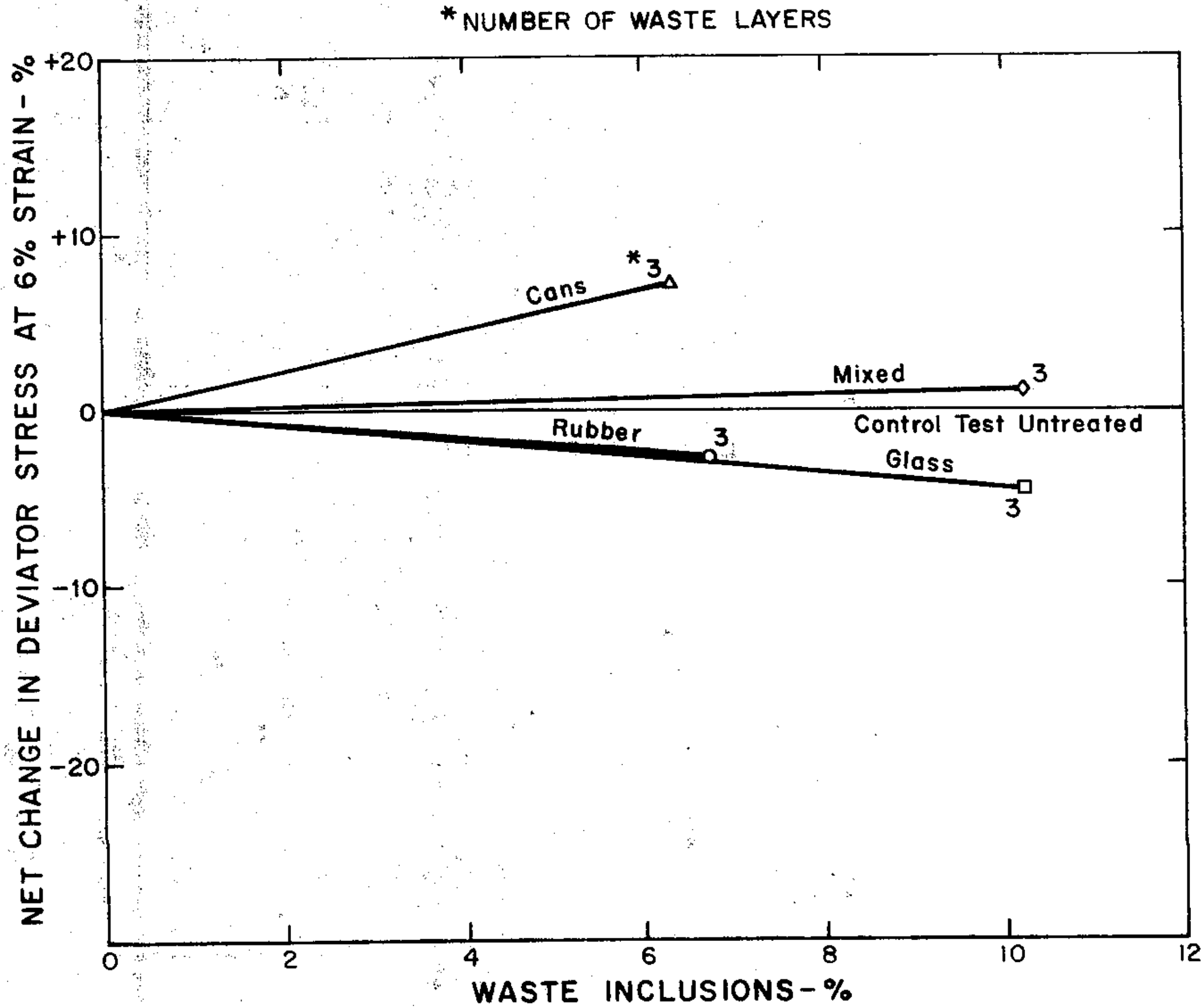


Figure 16

STRESS-STRAIN RELATIONSHIP
FOR TREATED AND UNTREATED SOIL

ALTAMONT #6 SOIL

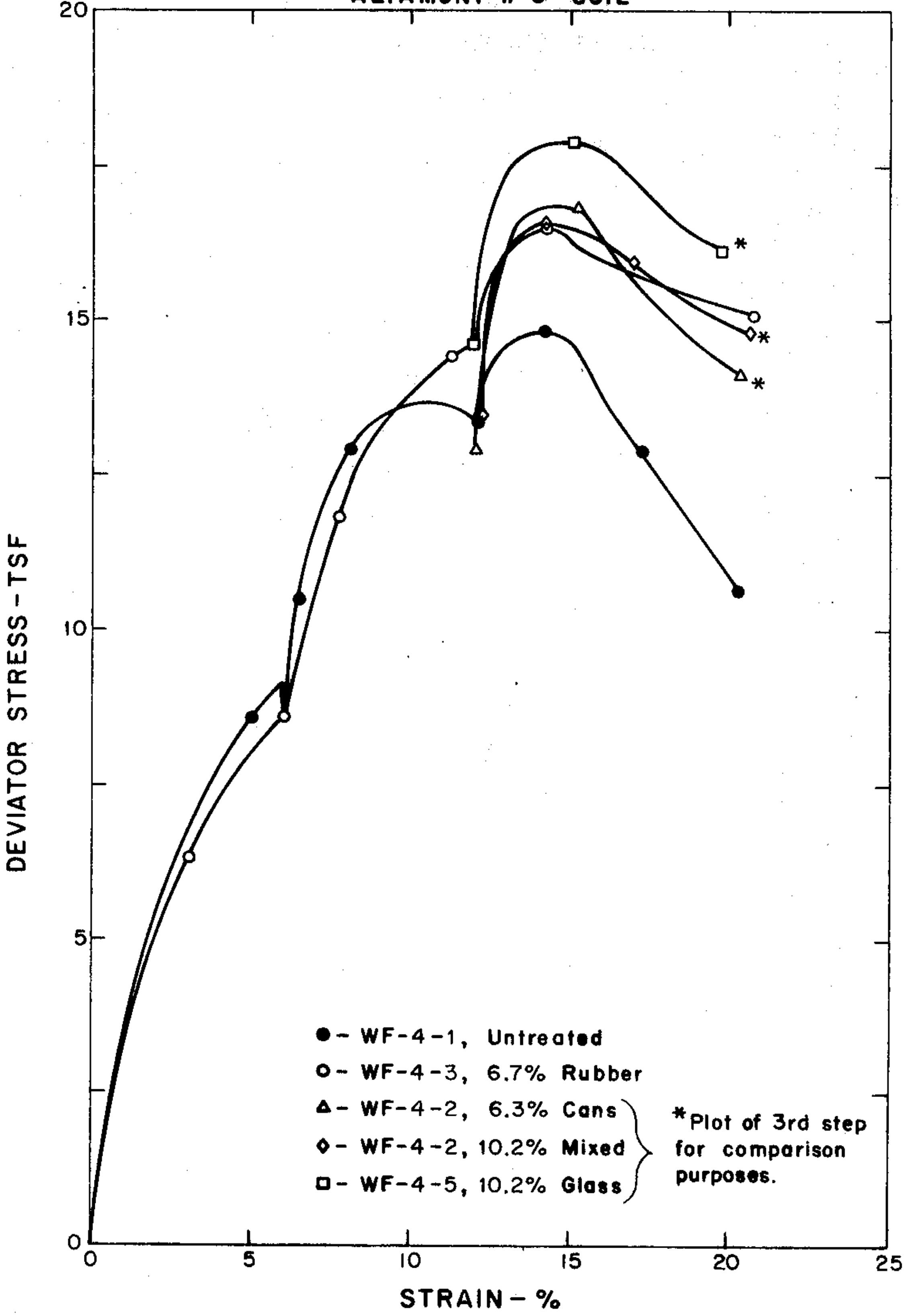
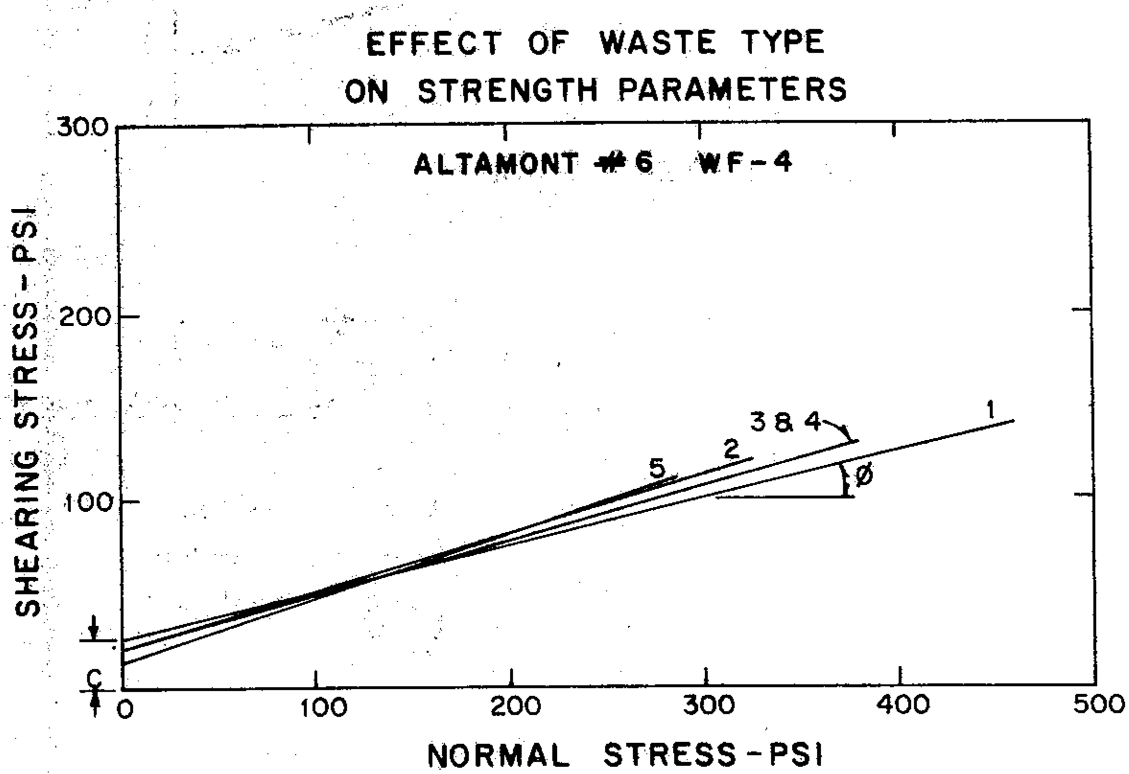


Figure 17



Envelope	Type of Waste	Percent Waste (% Dry Wt. of Soil)	Number of Layers	After Test Moisture (%)	c (PSI)	Ø (Deg)	Test Number
1	-	-	-	16.4	27	14.0	WF-4-1
2	Cans	6.3	3	16.4	21	17.0	WF-4-2
3	Rubber	6.7	3	16.1	21	16.0	WF-4-3
4	Mixed	10.2	3	16.4	21	16.0	WF-4-4
5	Glass	10.2	3	16.9	14	18.5	WF-4-5

TABLE 1

INDEX PROPERTIES OF EMBANKMENT SOILS

Description	SOURCE Discovery Park WF-1	 Aromas Red Sand WF-2	 Altamont Pass #8 WF-3	 Altamont Pass #6 WF-4
Identifying properties	Brown, slightly plastic clayey silt	Brownish-red, nonplastic silty sand	Tan, moderately plastic, clayey silt	Tan, moderately plastic, slightly silty clay
Soil Classification (Unified System)	ML	SM	CL	CL
Liquid limit	36	-	30	40
Plastic limit	27	-	18	23
Plasticity index	9	NP	12	17
R-value	39	63	31	25
Compaction (California Impact)				
Maximum Density (pcf)	107.2	129.7	122.7	119.6
Optimum Moisture (percent)	17.5	10.0	11.6	16.4
Strength Data				
Cohesion, $\bar{c}$, (psi)	0	0	5	27
Angle of internal friction, ϕ, (degrees)	24	33	28	14

TABLE 2

GRADING OF
RUBBER TIRE SCRAP

Sieve Size	Percent Passing Total Sample	Percent Passing As Used
6"	100	
3"	81	100
2 1/2"	65	80
2"	39	48
1 1/2"	14	17
1"	0	0

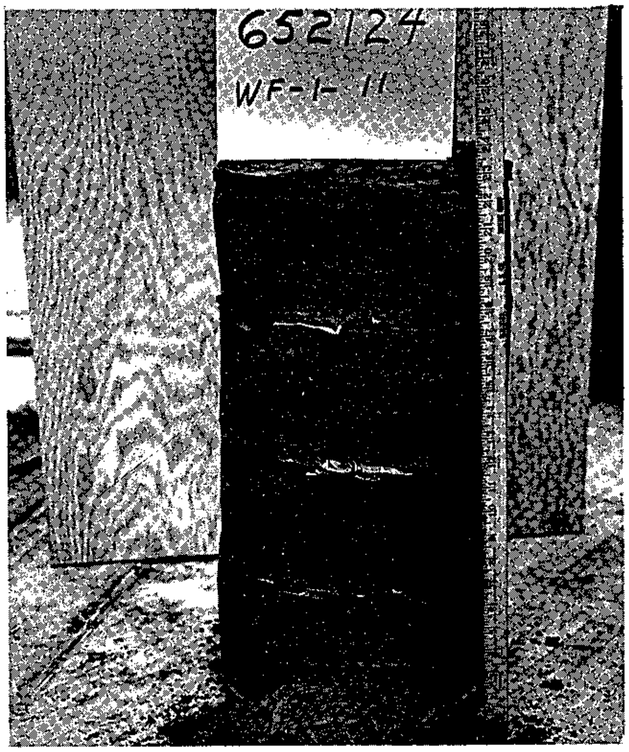


Photo 1 - Specimen WF-1-11 After test with total of 6.4% flattened tin and aluminum containers in 3 layers. (c = 0 & Ø = 25°)

Photo 2 - Specimen WF-2-4 After test with total of 6.8% chopped rubber tires in 3 layers. (c = 0 & Ø = 30°)

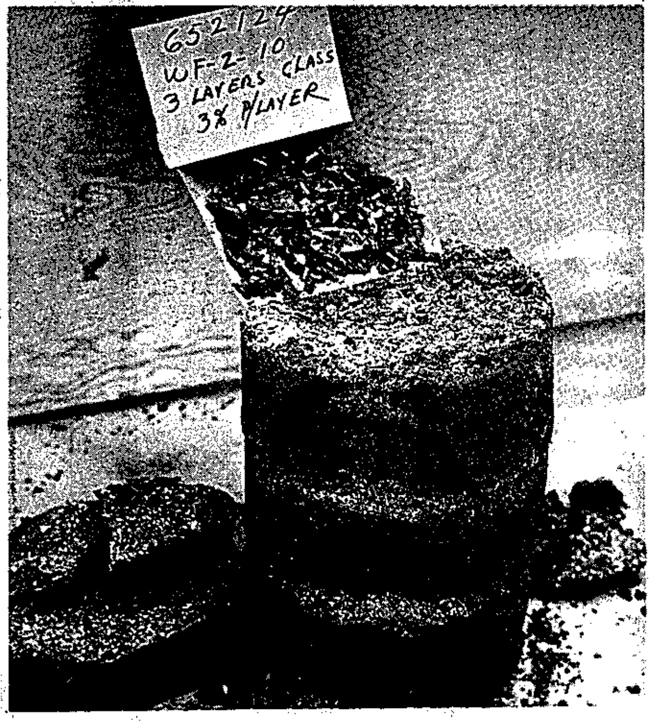


Photo 3 - Specimen WF-2-10 after test with total of 9.3% broken glass in 3 layers. Note condition of glass before and after test. ($c = 0$ & $\phi = 31.5°$)

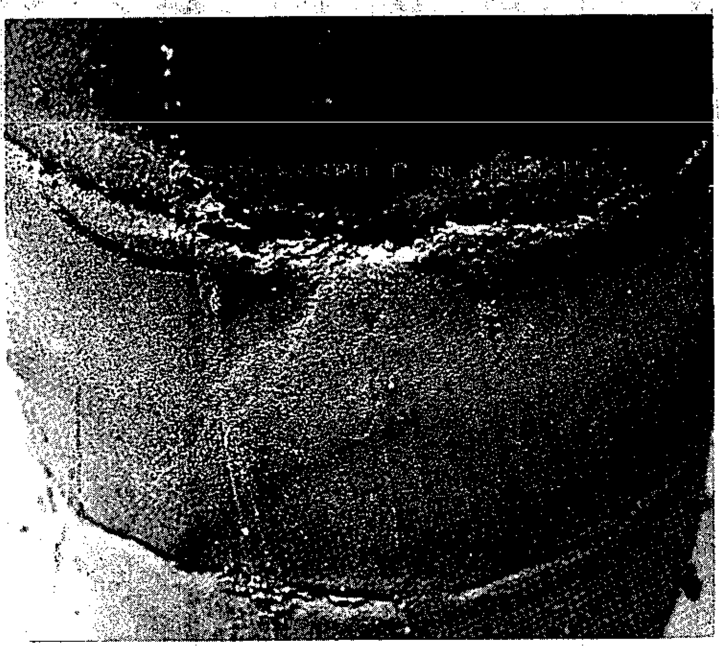


Photo 4 - Failure plane developed during testing of specimen WF-2-7 with total of 4.1% flattened tin and aluminum containers in 2 layers. ($c = 14$ psi & $\phi = 34°$)

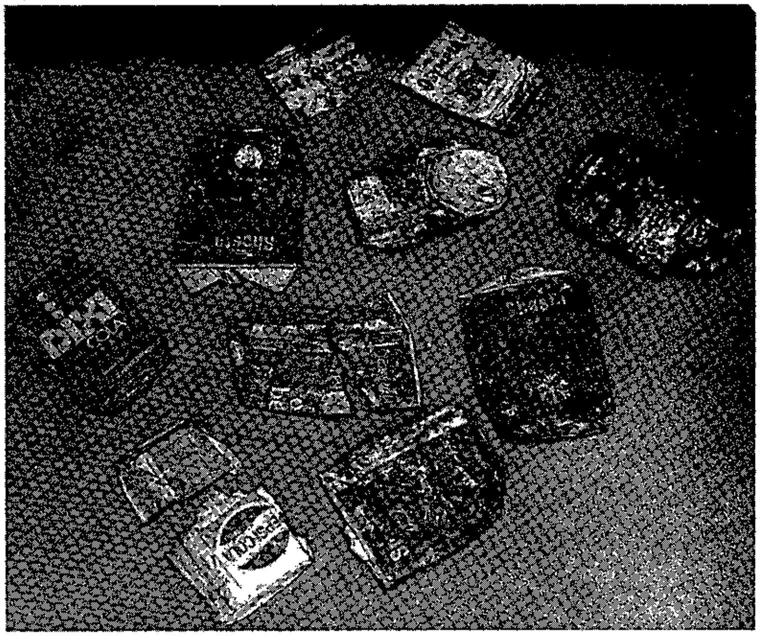


Photo 5 - Sheared cans removed from specimen WF-2-7 following tri-axial test.

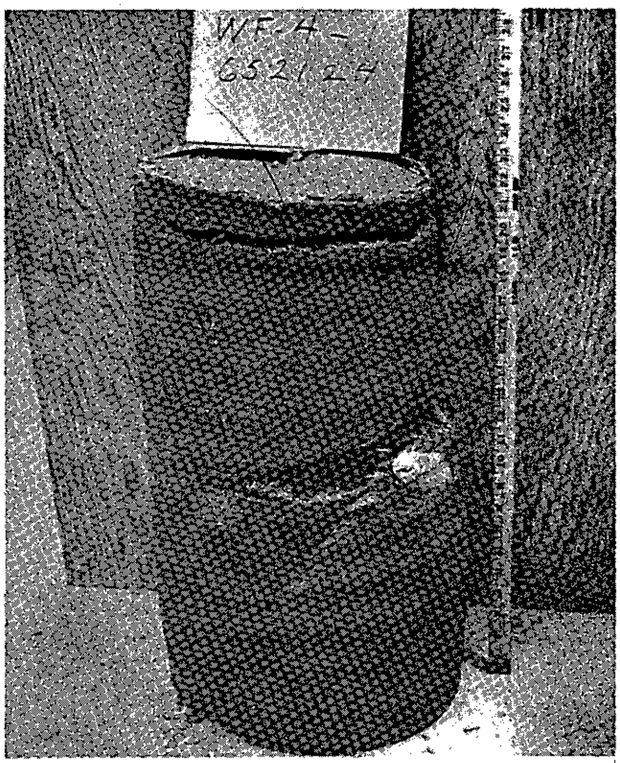


Photo 6 - Failure plane developed during testing of untreated specimen WF-4-1 (c = 27 psi & ϕ = 14°)

APPENDIX

METHOD OF TEST FOR LARGE-SCALE TRIAXIAL COMPRESSION OF SOILS

Scope

The large-scale triaxial compression test permits testing of soils containing particle sizes passing the 3-inch screen, and is used to test materials which are beyond the size limits for the smaller triaxial specimen. At least three specimens having similar soil characteristics are tested under different confining pressures. The plotting of stress-strain curves and construction of a Mohr envelope are performed in essentially the same manner as prescribed for the smaller triaxial testing procedure. See Test Method No. Calif. 230.

PART I. PREPARATION OF TEST SPECIMENS

A. Apparatus

The assembled apparatus is shown in Figure No. I. It is suitable for testing 12-inch diameter by 26.5-inch high remolded specimens. If the inner liner is used for soils containing jagged particles, the diameter is reduced to 11.75-inches. The major components are identified in Figures I and II.

1. *Pressure Chamber.* The pressure chamber is used to house the test specimen under pressure during the test. The complete chamber assembly consists of the following listed parts:

a. Base, with supply and pressure sensing hoses and lines.
b. Cylinder, with attached valve panel.
c. Head, with integral hydraulic loading ram and strain indicator dial.
d. Seals and assembly bolts.
e. Specimen membrane and clamps.
f. Specimen loading cap.
g. Solid lucite discs.
h. Porous stones.
i. Connecting lines.

2. *Control Console*—The console houses a hydraulic pump for applying test load, an air-operated water booster pump, and related regulators, valves and pressure gages for control and indication of confining and saturation pressures. Graduated pressure tanks permit application of confining and saturation pressures by house air or nitrogen pressure. The water booster pump provides an alternate source for confining pressure only.

3. *Auxiliary Equipment* (Figures III and IV)

a. Batching Pan
b. Specimen Mold Assembly
c. Specimen height gage
d. Tamping hammer
e. Scales, bucket and funnel
f. Leveling disc
g. Sprayer
h. "Torpedo" Level
i. Meter Stick
j. Miscellaneous items such as spatula, large screwdriver, moisture pans, etc.

B. Preparation of Specimen

1. The normal total sample weight will be approximately 2000 pounds. Separate the sample into the screen sizes shown on Figure V.

2. Establish the maximum dry density by performing Part II of Test Method No. Calif. 216. Use this data to determine the specimen dry weight and moisture content.

3. Prepare a Batch Computation Sheet, as shown in Figure V, to determine the total wet weight of specimen to provide desired density.

4. Weigh the required amount of each size material into the batching pan, and add required amount of water from sprayer. Add water while mixing material with a hoe or shovel. Cover the soil with a plastic sheet, and allow to hydrate overnight.

C. Test Record Forms

1. Use Form HMR T-2144-I, Figure VI, to record all data during shear testing of specimen.

PART II. UNCONSOLIDATED-UNDRAINED (UU) TEST METHOD

Scope

Test the specimen at the moisture content established during the batching operation. No drainage or consolidation is permitted during axial loading. Pore pressure is not measured nor considered in computing soil strength.

A. Assembly of Apparatus and Fabrication of Specimen

1. Solid lucite discs are used at specimen ends to prevent loss of moisture from the specimen during compression.

2. Erect the membrane, specimen mold and height gage.

3. Fabricate the specimen in six equal lifts, use both the lift height and the lift weight to control compaction. Weigh out one-sixth of the total wet specimen weight for the first lift. Pour approximately half of it into membrane. Stir the soil with a large screwdriver or similar tool to arrange particle distribution as evenly as possible. Tamp with tamping hammer to desired density using predetermined height to control density. Scarify the surface thoroughly, and tamp remainder of lift to specified height. Determine lift heights by using a meter stick to measure from lower edge of the gage bar to top of each lift. These predetermined distances are 68.8, 57.6, 46.4, 35.2, 24.0 and 12.8 cm respectively. Scarify each tamped surface before adding more soil. Tamp the final lift to within a few millimeters of required height, then bring to final dimension by use of levelling disc. Check with torpedo level. The specimen will be the correct height when surface of levelling disc is 10.3 cm from lower edge of gage bar. Remove levelling disc and install lucite disc.

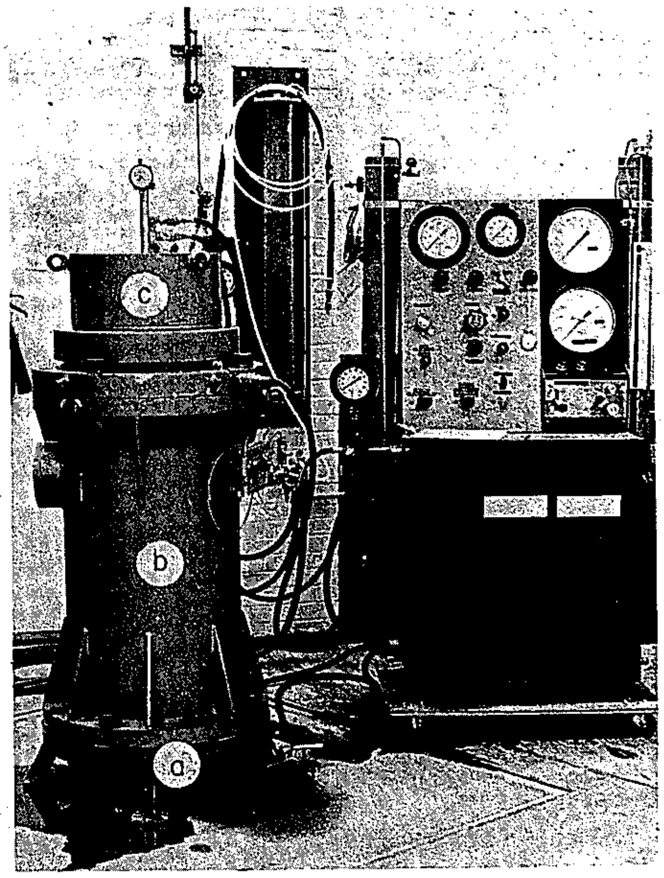


FIGURE I

ASSEMBLED 12-INCH TRIAXIAL TEST CHAMBER AND CONTROL CONSOLE

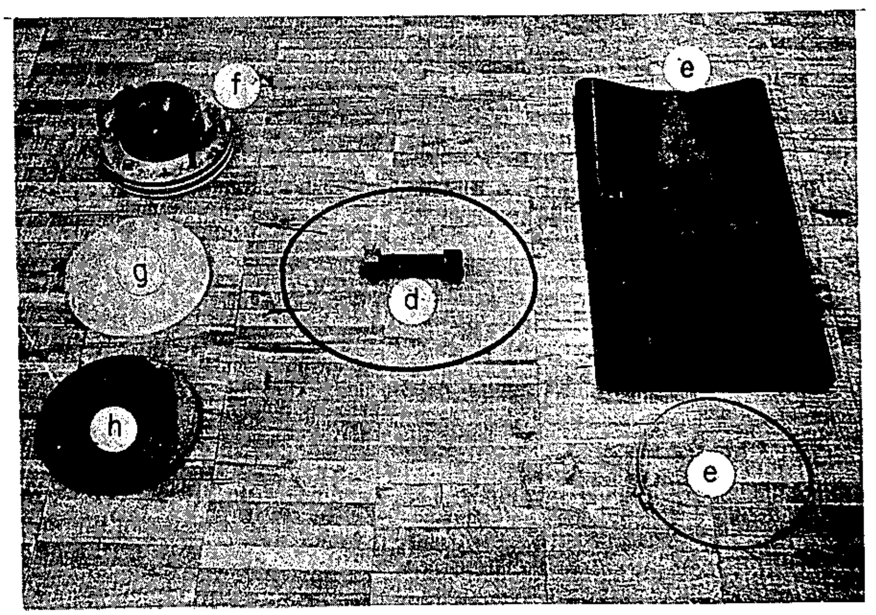


FIGURE II

COMPONENT PARTS OF TEST CHAMBER ASSEMBLY

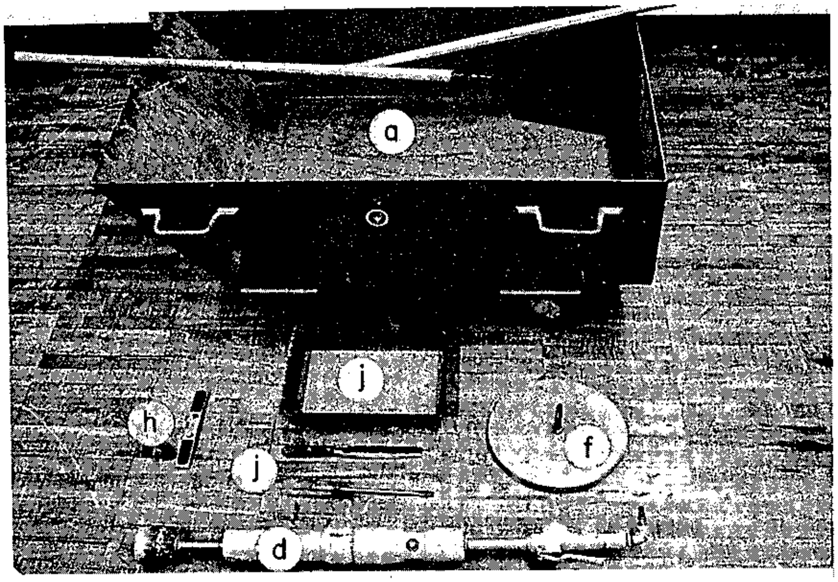


FIGURE III
SAMPLE BATCHING COMPONENTS

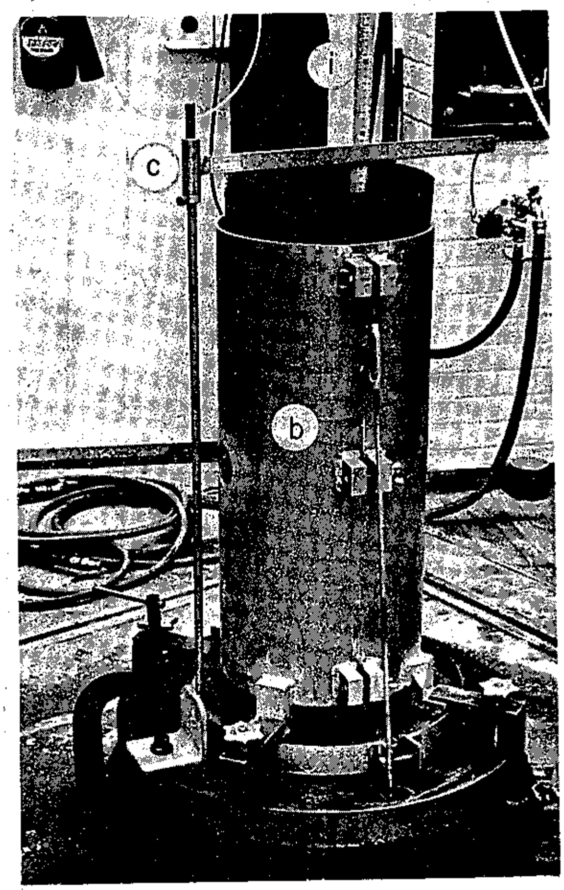


FIGURE IV
SPECIMEN MOLD AND HEIGHT GAGE
ASSEMBLED ON BASE

BATCH COMPUTATIONS FOR 12" TRIAXIAL SPECIMENS

Max. Dry Unit Weight: 127.9 Lbs. Volume of Specimen: 1.6629 Job: Pine Creek

Optimum Moisture : 11.9 % Dry Batch Weight : 197.9 Lbs. Sample: PC-8

Batch Unit Weight @ 93% RC : 119.0 Batch Moisture : 221.4 Lbs. Date: 7 Dec. 70

Screen Size	Total Sample % Passing	Adjusted % Passing	% Retained on Indicated Screen	Dry Batch Weights, lbs.	Sack Moisture	Adjusted Sack Batch Weights, lbs.
3	91	100	0	0	4.2 %	0
2-1/2	89	98	2	4		4.1
2	87	96	2	4		4.1
1-1/2	85	93	3	5.9		6.0
1	80	88	5	9.9		10.1
3/4	75	82	6	11.9		12.2
1/2	68	75	7	13.9		14.2
3/8	61	67	8	15.8		16.2
#4	48	53	14	27.7		28.5
-#4			53	104.9		108.4

Water Required: 23.6 Lbs.

Water in Soil : 5.8 Lbs.

Water to Add : 17.8 Lbs.

Moisture After Test: ______

FIGURE V

MATERIALS AND RESEARCH DEPARTMENT
12" TRIAXIAL COMPRESSION TEST DATA

Date____ Test By____ Chamber No.____ Transd. No.____ Station No.____
Type of Test CD ☐ CU ☐ UU ☐ Strain Rate________in/min.
% Compaction____ Target Moist.____ Actual Moist.____ Elev.____ Depth____

Job Name____________ Conf. Press σ_3 ____________
Job No.____________ Nominal Ht.____________
Sample No.____________ Initial Ht.____________
Specimen No.____________ Change in Ht.____________
Chamber Press(σ_C)____________ Initial Diam.____________
Back Press____________

% Strain	Dial Reading	Axial Load	Top Pore Press	Cent. Pore Press	Bot. Pore Press	Chamb. Vol.	Spec. Vol.	Drawing of Specimen
0	0							
1/4	0.067							
1/2	0.133							
3/4	0.199							
1	0.265							
1-1/2	0.398							
2	0.530							
2-1/2	0.663							
3	0.795							
3-1/2	0.927							
4	1.060							
4-1/2	1.193							
5	1.325							
6	1.590							
7	1.855							
8	2.120							
9	2.385							
10	2.650							
12	3.180							
14	3.710							
16	4.240							
18	4.770							
20	5.300							

Remarks:________________________________

HMR T-2144-I (9/71)

FIGURE VI

B. Test Procedure

1. Install specimen loading cap and clamps. Assemble pressure chamber and connect hydraulic and water hoses from console.

2. Bring ram into contact with loading cap, and record strain indicator dial reading to nearest 0.001 inch in "Initial Height" block of test form. Back ram off three tenths of an inch.

3. Fill chamber and pressurization tank. Pressurize to desired confining pressure and allow specimen to compress for five minutes. Start ram and adjust to 0.133 inch per minute travel. Read ram pressure at that rate and record initial pressure as first entry in Axial Load column at 0% strain on test sheet. Continue ram advance until contact with specimen, then stop ram. Reset strain indicator dial to zero. Set chamber tank burette scale to zero. Tank should be approximately half-full. Adjust level prior to starting load test.

4. Check chamber pressure, and start ram. Record ram load at specified intervals of strain, and immediately check and record chamber volume change as plus or minus quantity from zero. Check strain rate frequently with a stop watch, and correct as needed. Keep chamber pressure regulated to proper value.

C. Disassembly

1. When specimen has been strained the required distance, reverse the ram and fully retract on fast advance setting. Stop the machine, bleed off residual hydraulic pressure, and disconnect hydraulic hoses. Release chamber pressure, and drain chamber and pressure tank. Dismantle chamber.

2. Remove all membrane clamps, loading cap and upper disc. Pad the sharp edge of chamber base to prevent membrane cuts. Gently tip membrane and specimen off base and lower to floor. Carefully remove material from membrane with the large pointed rod. *This is a dangerous operation and the operator must insure proper working clearance for himself, and adequate precautions for other personnel in the area.* There is also the risk of puncturing the membrane.

3. Select a representative sample of approximately 10,000 grams from the specimen for checking moisture content. Refer to Test Method No. Calif. 226. Record "after test" moisture (A.T.MOIST.) on test form.

D. Calculations and Plotting

1. The test forms for a completed series of tests are routed to the computer for processing. Appropriate graphs and Mohr envelope are plotted from finished test data. See Figures VII, VIII, IX and Appendix 1 of Test Method No. Calif. 230.

PART III. CONSOLIDATED UNDRAINED TEST (SATURATED) WITH PORE PRESSURE MEASUREMENT (C-U)

Scope

For this type test, the specimen is first saturated, then consolidated at the desired net confining pressure. During the straining operation, no drainage is permitted from the specimen, and measurement is made of pore water pressure generated within the specimen.

A. Apparatus

1. Use the porous stones at each end of specimen.

2. This type test requires installation of the center pore pressure probe and connection of water lines to loading cap and chamber base.

3. The pore pressure indicating device is used to measure pore pressure at either end or center of specimen.

4. Water for saturation is supplied from the graduated saturation reservoir on left side of control console, which also receives water from the specimen during consolidation.

B. Test Record Forms

1. Use Form HMR T-2144-I, Figure VI, to record all data during shear testing of specimen.

2. Use Form HMR T-2145 (See Figure VI of Test Method No. Calif. 230) to record saturation and consolidation data.

C. Preparation of Specimen

1. See Part I.

D. Assembly of Apparatus

1. After membrane is erected on base, add approximately one inch of water inside membrane. Open both valves at outer end of lower pore pressure and saturation lines on base. Allow water to flow until free of bubbles, then close valves. Wet lower porous stone and insert in membrane, insuring full bottoming on base. Soak until ready to tamp material into place.

2. Immediately prior to tamping soil into membrane, drain water from base until surface of stone is free of standing water.

3. Tamp material into place as described in Part II, Section A, until the third lift is completed and the specimen midpoint at 46.4 cm is reached. Scarify surface thoroughly.

4. Insert, connect and bleed the center pore pressure probe.

5. Tamp in remaining three lifts of specimen, install upper stone, loading cap and clamps.

6. Connect flexible lines to loading cap and center pore pressure probe. Caution: Insure that valve on center pore pressure sensing line is open, since it will be inside the assembled chamber.

7. Assemble chamber, and connect outside lines from panel to base. Connect and bleed the pore pressure indicator. Insure that valving configuration is such that no water is admitted to the specimen during this time.

E. Test Procedure

1. Establish ram contact with the specimen, and read initial height on strain indicator dial gage to

nearest 0.001 inch. Record on test form. Retract ram three-tenths inch.

2. Fill chamber and chamber pressurization tank. Apply 5 PSI confining pressure to the specimen.

3. Adjust water level in saturation tanks to full and set burette scale at a chosen quantity indication.

4. Position valving so that water is admitted into the specimen from bottom to top, and drainage permitted from top to exterior of chamber. Place end of vent line in a one-gallon plastic jug.

5. Permit head pressure saturation until water quantities into and out of the specimen are equal. Bleed across both stones until water is free of bubbles. Close the vent line, and start back pressure saturation.

6. Back pressure saturation is used to force the air in the specimen voids into solution. It consists of raising chamber pressure and saturation pressure in 10 PSI increments. The consecutive steps at each pressure increase are as follows:

a. Close all valves on panel which would permit flow of water from the specimen.

b. Note center pore pressure.

c. Raise chamber pressure 10 PSI.

d. Recheck pore pressure.

e. Divide pore pressure increase from initial reading by quantity of increase made in chamber pressure, and record in "B Parameter" column of Form HMR T-2145 (See Figure VI of Test Method No. Calif. 230). Example: Chamber pressure was raised 10 PSI. Pore pressure increased 5 PSI. $\frac{5}{10}$ = "B" parameter of 0.5.

f. Set saturation pressure at 5 PSI below chamber pressure, and apply to both ends of specimen.

g. The above sequence is followed until a "B" parameter of at least 0.96 is attained. Back pressure in excess of 100 psi will frequently be required. In cases where difficulty is encountered in attaining saturation, it will sometimes help to carefully bleed across both stones to eliminate air pockets. Set valving so that pore pressure indicator is registering saturation pressure, and control rate of bleeding so that pressure drops no more than 1 PSI.

7. When a "B" parameter of at least 0.96 has been attained, the specimen shall next be consolidated at the net confining pressure required for the test.

8. Close valving to isolate specimen from saturation pressure tank.

9. Drain tank until one-third full and set scale to zero. Readjust pressure to the last used saturation pressure value.

10. Raise chamber pressure to an amount equal to net confining pressure plus saturation pressure. Example: Net desired confining pressure 100 PSI, last used saturation pressure was 50 PSI. Total chamber pressure 150 PSI.

11. Recheck "B" parameter for a value of at least 0.96. Failure to attain this value will require reduction of chamber pressure and a return to saturation procedure. If parameter is satisfactory, continue with consolidation of specimen.

12. It is desirable to use two operators during the first few minutes of consolidation; one to time and record consolidation output, the other to adjust and maintain chamber and back pressures.

13. During consolidation pore water within the specimen is permitted to flow into the back pressure tank by opening valves that permit drainage from both ends, but not the center.

14. Consolidation is complete when center pore pressure has equalized with back pressure, and no further rise is noted on tank scale. The specimen is now ready to be sheared.

15. The rate of shear must be slow enough to permit nearly equal pore pressure throughout the specimen. The degree of permeability will have been indicated by the time lapse for equalization of center and end pressures within the specimen during saturation and consolidation. The standard rate of shear for permeable material is 0.022 inch per minute for 241 minutes, and for a less permeable specimen, the rate is reduced to 0.011 inch per minute for 482 minutes. At the faster rate the operator must closely observe and compare the pressure between center and ends of the specimen during the first two minutes of shear. A difference of more than 2 PSI or an increasing differential in pressure indicates an excessive strain rate, which should be reduced to 0.011 inch per minute.

16. Start the ram, adjust to desired rate, and record ram pressure at that rate and chamber pressure. Continue until contact is made with specimen. Stop ram, record difference in height, and reset strain indicator dial gage and tank burette to zero.

17. During shearing operation, record ram load, pore pressure and chamber volume change. When specimen permeability is such that pore pressure at ends and center is more than 2 PSI in difference, record each pressure separately on test form. Insure that valving is positioned so that only one point in the specimen is indicated.

18. If strain rate was changed after recording initial ram pressure reading, a corrected reading must be obtained after shear is completed. Retract the ram until well clear of specimen, then advance at test rate against chamber pressure used during test. Allow ram pressure to stabilize, and record as corrected initial pressure.

F. Disassembly

1. Upon completion of shear, dismantle equipment and remove specimen as described in Part II.

PART IV. CONSOLIDATED DRAINED TEST (CD)

Scope

In this type of test, the specimen is saturated, then consolidated and sheared with drainage permitted from the specimen.

A. Apparatus

1. Use the same apparatus as described in Part III.

B. Test Record Forms

1. See Section B of Part III.

C. Preparation of Specimen

1. See Part I.

D. Assembly of Apparatus

1. See Section D of Part III.

E. Test Procedure

1. Use procedure in Part III, with the following exceptions.

a. Upon completion of consolidation, the specimen is left open to drainage. Further volume change during shear is recorded in "Spec. Vol." change column of Test Form HMR T-2144-I, Figure VI.

b. Strain rate must be slow enough to prevent formation of pore pressure in excess of 2 PSI above consolidation back pressure. In event soil characteristics do not permit drainage at the slowest rate, center pore pressure will be recorded.

F. Disassembly

1. As described in Part II.

REFERENCES

A California Method
Test Methods Nos. Calif. 216, 226 and 230

End of Text on Calif. 232-A